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FROM THE AMERICAN PEOPLE

ENGINEERING SUPPORT PROGRAM

WORK ORDER WO-LT-0077 AMD3
GARDEZ TO KHOST ROAD, BRIDGE No.10
SCOUR ANALYSIS AND FOUNDATION STUDY

July 28, 2014

This publication was produced for review by the United States Agency for International Development. It was prepared by Tetra Tech, Inc.

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TETRA TECH

July 28, 2014

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Re: USAID Contract No. EDH-I-00-08-00027-00 / Task Order No. 1
Afghanistan Engineering Support Program (AESP)

Work Order WO-LT-0077 AMD3
Gardez to Khost Road Bridge No. 10
Scour Analysis and Foundation Study

Dear [redacted]:

It is with great pleasure that Tetra Tech submits the Scour Analysis and Foundation Study for the Gardez to Khost Road Bridge No. 10.

As summarized in the Executive Summary of this report, Tetra Tech's analysis resulted in the determination that the 2010 Design performed by others does not provide adequate scour protection, does not meet AASHTO LRFD stability requirements and does not provide adequate life safety in a seismic event. In addition, the 2010 Design roadway profile does not provide adequate drainage and will result in flooding on the bridge. For these reasons, Tetra Tech recommends that the Bridge No.10 crossing be redesigned.

We look forward to supporting the USAID OEGI mission during 2014 and to strengthen our partnership while building a brighter future for Afghanistan.

Please contact me at your convenience should you have any questions or comments regarding this report.

Respectfully,
Tetra Tech, Inc.

[redacted] BCEE
Chief of Party (AESP)

AFGHANISTAN ENGINEERING SUPPORT PROGRAM

Contract No. EDH-I-00-08-00027-00
Task Order No. 1
Work Order WO-LT-0077 AMD3

GARDEZ TO KHOST ROAD BRIDGE No. 10 SCOUR ANALYSIS AND FOUNDATION STUDY

JULY 28, 2014

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1.0 Executive Summary

Bridge #10 was located on the Gardez to Khost Road in Afghanistan, spanning over a tributary immediately west of a main river. As part of the overall Gardez to Khost Road reconstruction project, The Louis Berger Group performed preliminary hydraulic modeling of the crossing. Based on this modeling, they determined that the bottom elevation of the superstructure was not sufficient to meet the hydraulic demands of the crossing. A complete bridge replacement was proposed and a two-span cast-in-place concrete slab bridge was designed (2010 Design).

Subsequently, the existing Bridge #10 was destroyed by floods and a temporary pipe culvert was installed. Prior to construction of the 2010 Design, USAID requested that Tetra Tech perform a topographical survey, geotechnical investigation, geotechnical analysis, hydraulic modeling and structural analysis in order to determine if the 2010 Design is in conformance with the latest AASHTO LRFD Bridge Design Specifications, 6th Edition, 2012 (AASHTO LRFD) standards and adequate based on the complete hydraulic, geotechnical and structural analyses.

Tetra Tech developed a hydraulic model of the 2010 Design based on the channel geometry as determined by the topographical survey and the gradation of the soils as determined by the geotechnical investigation. The hydraulic model confirmed that the 2010 Design provided an adequate hydraulic opening and has a bottom of superstructure elevation that allows adequate freeboard over the 50-year design flood elevation. However, the scour analysis resulted in large scour cavities at the abutments and the pier (approximately 10 m deep maximum). **The 2010 Design is not adequate to withstand scour.**

Tetra Tech performed geotechnical analyses based on the three borings and two test pits performed during the geotechnical investigation, and the applied loads from the 2010 Design, as calculated by Tetra Tech. The geotechnical calculations, performed in accordance with AASHTO LRFD resulted in calculated settlements less than 10 mm and an ultimate bearing resistance less than required for the abutments under seismic loading. Based on the structural stability calculations performed by Tetra Tech, the **2010 Design for the abutments and piers fail to meet the AASHTO LRFD stability requirements for bearing resistance, overturning or sliding.**

In addition, the 2010 Design does not include cheekwalls on either the abutments or the pier (which normally are used for restraint of the superstructure in a seismic event). Instead, it uses dowel bars to tie the superstructure and substructure together at the bearing locations. Therefore, the 2010 Design does not allow relative movement between the superstructure and substructure (either longitudinal or transverse movement, even due to thermal expansion). **It should be expected that the 2010 Design would result in increased cracking and maintenance over the life of the structure, and may not meet life safety requirements in a seismic event.**

In order to address these issues, along with issues with the roadway profile and drainage, **Tetra Tech recommends that the Bridge #10 crossing be redesigned**. The estimated associated costs for the recommendations are included herein.

2.0 Introduction / Purpose

Bridge #10 was located on the Gardez to Khost Road in Afghanistan, spanning over a tributary immediately west of a main river. As part of the overall Gardez to Khost Road reconstruction project, The Louis Berger Group performed preliminary hydraulic modeling of the crossing. Based on this modeling, they determined that the existing bridge had insufficient hydraulic capacity to pass the 50-year peak discharge. A complete bridge replacement was proposed and a two-span cast-in-place concrete slab bridge was designed (2010 Design).

Subsequently, the existing Bridge #10 was destroyed by floods and a temporary pipe culvert was installed. Prior to construction of the 2010 Design, USAID requested that Tetra Tech perform a topographical survey, geotechnical investigation, geotechnical analysis, hydraulic modeling and structural analysis in order to determine if the 2010 Design is in conformance with the latest AASHTO LRFD Bridge Design Specifications, 6th Edition, 2012 (AASHTO LRFD) standards and adequate based on the complete hydraulic, geotechnical and structural analyses.

3.0 Field Work

3.1 Topographical Survey

GeoTechnique Company (GTC) performed the site survey in April 2014 and summarized their findings in a report entitled "Survey Report for Topographical Survey for Gardez to Khost Bridge No. 10, Afghanistan" dated 01 May 2014. The limits of survey included approximately 500m of length and 120m of width, capturing both roadway approaches to the bridge, the tributary channel which Bridge #10 crosses and the eastern bank of the main river immediately east of the proposed bridge crossing. GTC's report summarizes their work and includes numerous site photos.

3.2 Geotechnical Investigation

The Geotechnical investigation was performed by Shawal Geotechnical Engineering/ Materials Testing Laboratory (Shawal GMTL). The Geotechnical investigation included borings, test pits, sampling, field testing and laboratory testing. A summary of the field investigation and the results of the testing are provided in a report entitled "Soil Test Results Reports for Gardez to Khost Bridge #10, Khost Province, Afghanistan" dated 29 May 2014.

As noted in their report, their investigation included three boreholes with a completion depth of 15 meters below the existing ground surface. One borehole was drilled at each of the bridge supports (abutment & pier footings) and two test pits were excavated in the channel. Laboratory analyses of the samples were also performed to evaluate engineering characteristics of the bridge's subgrade.

Soil samples were obtained during the drilling operations by driving a Standard Split Spoon sampler at 1 meter intervals. The sampler was driven with a 140-pound hammer free falling 30 inches. The number of blows required to drive the sampler were recorded in accordance with the Standard Penetration Test (SPT) per ASTM D1586. The SPT values are useful in evaluating the relative density and consistency of the soils. The SPT values indicated the alluvial soils generally range from medium dense to very dense. In some cases, refusal was listed at areas logged as boulders. The soil samples recovered during the drilling operations were tested for in-situ moisture content, Atterberg Limits, and gradation. In addition, one soil sample from each boring was subjected to direct shear strength testing per ASTM standard D3080.

Larger bulk soil samples were obtained from the test pits excavated in the channel. These samples were tested for in-situ moisture, modified Proctor moisture / density relationships, California Bearing Ratio on samples compacted to 95% of modified Proctor density, and gradation analyses. In addition, in-situ moisture and density were measured at each test pit using sand cone methods. Gradation testing on the test pit samples is considered more representative due to the coarseness of the alluvium.

The Shawal GMTL report reflects that the subsurface material is non-plastic to low plastic and medium dense to very dense, generally coarse alluvium. The alluvial clasts range in size from sand to cobble and boulder sized material and are locally silty and/or clayey. Groundwater was encountered approximately 4.0 m below the channel bed.

4.0 Geotechnical Evaluation

4.1 Review of Geotechnical Data

Although the geotechnical report prepared by Shawal GMTL contained geotechnical design parameters and recommendations, Tetra Tech independently performed calculations to determine the design parameters and recommendations in accordance with AASHTO LRFD since the subsequent bridge evaluation (see Section 6.0) was performed in accordance with AASHTO LRFD.

Tetra Tech's full recommendations, including ultimate bearing resistance calculations and a settlement analysis, can be found in "Engineering Support Program, WO-LT0077, Gardez to Khost Road, Bridge #10, Geotechnical Report" dated June 17, 2014. These calculations were based on soil property values that are typical of those soils encountered in the soil boring logs, the gradation analysis of the test pits performed in the channel and the following assumptions:

- Used AASHTO LRFD methodology considering the shape of the foundation, depth of embedment, and the shearing resistance of the soil above the foundation.
- Assumed bearing soil is fully saturated
- Assumed cohesion value is zero since the soils encountered underlying the bridge foundation are granular and non-plastic in nature.
- Used footing geometry as defined in the 2010 Design

4.2 Recommended Design Parameters

Tetra Tech performed geotechnical analyses based on the three borings and two test pits performed during the geotechnical investigation, and the applied loads from the 2010 Design, as calculated by Tetra Tech. The geotechnical calculations, performed in accordance with AASHTO LRFD resulted in calculated settlements less than 10 mm.

Tetra Tech recommends that Bridge #10 be supported on shallow foundations (spread footings) at the abutments and piers. The bottom of footings shall be located a minimum of 1.0 m below grade due to frost concerns.

The abutments and pier should be designed in accordance with the following design parameters:

- Groundwater level at channel grade
- Weight of Soil = 20.5 kN/m^3 (130.4 pcf)

- Factored Bearing Resistance for Pier:
  - Non-Seismic Load Cases (ø=0.45) 561 kN/m^2 (11.75 ksf)
  - Seismic Load Cases (ø=1.0) 1246 kN/m^2 (26.00 ksf)
- Factored Bearing Resistance for Abutments:
  - Non-Seismic Load Cases (ø=0.45) 381 kN/m^2 (8.00 ksf)
  - Seismic Load Cases (ø=1.0) 847 kN/m^2 (17.78 ksf)
- Angle of Internal Friction = 33 degrees

- Ko = 0.46
- Ka = 0.29
- Kp = 3.39
- Coefficient of Friction for Sliding = 0.57

5.0 Hydraulic Evaluation

5.1 General

At the Bridge #10 crossing, the Gardez to Khost Road crosses a tributary immediately west of a main river. Based on the topographic mapping, both the tributary and the main river are steep (average of 1% and 3%, respectively). Geotechnical data shows that the riverbed material is granular and non-plastic. Photographs in the survey report depict the river and tributary as braided, and cobble dominated systems with high width to depth ratios. It is possible these systems have high sediment supply, with the potential for excessive deposition both longitudinally and transversely. The banks appear to be erosive, likely a result of lateral movement of the river in response to significant flows. There are small settlements or individual homes located along the banks.

5.2 Hydraulic Model

The 2010 Design hydrologic analysis for the 50-year flood was supplied to Tetra Tech for use in this analysis. This data was supplied as a report excerpt with no documentation of calculation parameters. Standards in the industry typically utilize the 100-year event for hydraulic parameters of the bridge and a ‘check design’ procedure based on the 500-year event. The hydraulic capacity of the bridge and scour potential were evaluated using the estimated peak 50-year discharge on the tributary as stipulated by the project scope. The 2010 Design hydrologic analysis reports a 50-year discharge used for this analysis was 185.30 m^3/s. The watershed area for the tributary was reported as approximately 115.30 km^2.

Two hydraulic scenarios were assessed: 1) analyses of the main stem with backwater effects on the tributary, producing the highest flow depths at the bridge, and 2) low flows in the main river and the 50-year discharge in the tributary producing the highest velocities calculated at the bridge. In addition, a hydraulic model of the main river was prepared to evaluate the scour potential at Bridge #10 due to the main river flow and other potential impacts on the bridge or the approaches. Using the hydrologic analysis in the 2010 Design report, the 50-year peak discharge of the main river was estimated. No peak discharge for the main river is reported at the location of Bridge #10, but peak discharge was reported for Bridges #9 and #11 which bracket the site.

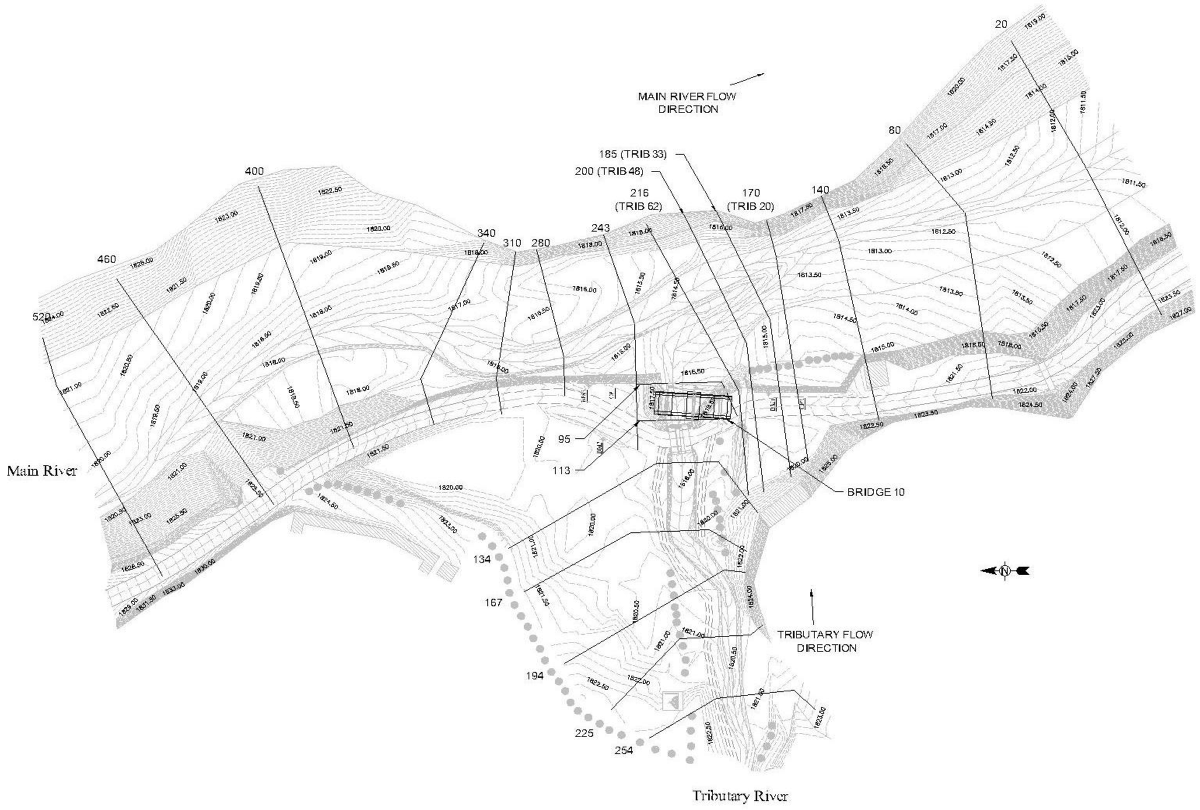
To estimate the peak flow of the main river, the discharge and area for each crossing were plotted on a graph and fitted with a linear regression line passing through the origin. Data was used only if it was reported as calculated using HEC-HMS. An equation for the linear regression line was determined by the computer, using area as the variable. Using Soviet-era topographic data and the data within the report, the total drainage area of the main river at Bridge #10 was estimated to be approximately 529.13 km^2. The estimated peak flow of the main river at Bridge #10 is approximately 820 m^3/s.

A hydraulic analysis was conducted using HEC-RAS version 4.1.0, encoded using the topographic survey. The main river model consists of fourteen cross sections encoded at an interval between 10 and 60 meters. A Manning's *n* value of 0.045 was selected to represent the rocky, largely unvegetated condition within the channel and the overbank areas.

Along the tributary, eleven cross sections were encoded at an interval between 10 and 30 meters. A Manning's *n* value of 0.045 was selected to represent the rocky, unvegetated conditions in the river channel as shown in site photographs. A Manning's *n* value of 0.06 was used to represent some areas of vegetation and agriculture on each overbank area upstream of Bridge #10.

The 2010 Design is a two-span cast-in-place slab bridge with one pier. Each span is approximately 12.17 meters long from pier/abutment centerline to pier/abutment centerline providing a total conveyance width of 24.34 meters. The road width of the bridge is approximately 8.0 meters. It was assumed for this analysis that the finished grade of the river bottom under the bridge will be approximately elevation 1816.31 meters upstream of the bridge and at approximately 1815.51 meters downstream of the bridge. The regraded elevations are located approximately 3 meters upstream and downstream of the bridge face, respectively. This is slightly lower than the existing channel grade, and is recommended to provide a continuous grade through the bridge.

In addition to grading in the vicinity of the bridge, a transition channel connecting the bridge opening and the existing channel is recommended. The transition channel would tie in the bridge opening to the existing channel at a point approximately 50 meters upstream. The transition channel lowers the effective slope of the creek. The channel would have a variable bottom width with 3:1 H:V side slopes.



Hydraulic modeling results show velocities at the bridge approach of 2.30 m/s and shear stresses of approximately 77.7 N/m^2. Velocities through the bridge range from 3.27 m/s to 4.34 m/s. Velocities downstream of the bridge remain high, as the tributary meets the main river. The maximum water surface elevation at the upstream face of the bridge crossing is approximately at elevation 1819.17 meters. A summary of HEC-RAS results for the tributary is presented in Table 1.

Table 1
Summary of HEC-RAS Calculations - Tributary

Cross Section	Water Surface El. (m)	Avg. Velocity (m/s)
20*	1815.17	3.11
33*	1815.48	2.86
48*	1815.66	3.06
62*	1815.95	2.61
95	1817.23	3.97
	Bridge	
113	1819.17	2.30
134	1819.35	4.45
167	1821.30	4.26
194	1822.07	3.14
225	1822.23	4.19
254	1823.10	3.67

* Coincident with main river

A separate hydraulic model was prepared for the main river to evaluate the potential scour effects on Bridge #10 and potential overtopping of the approach roads or bridge. Modeling results for the main river showed that the approach roads and bridge have sufficient elevation above main river 50-year flood elevations. Regarding scour potential, evaluation of the model and topographic survey shows that the bridge location is outside the main flow areas of the river. This isolation from the main flow normally creates an ineffective flow area, which is characterized by very low flow velocities. The excavated channel flowline elevation is also higher than the main river, creating shallower flooding depths in the ineffective area. The scour potential at the bridge due to the main river is expected to be no greater than the scour potential due to the tributary flow.

A summary of the HEC-RAS results for the main river is presented in Table 2.

Table 2
Summary of HEC-RAS Calculations – Main River

Cross Section	Water Surface El. (m)	Avg. Velocity (m/s)
20*	1814.32	4.48
80*	1815.16	5.34
140*	1816.24	5.91
170*	1816.84	5.82
185*	1817.47	4.22
200*	1817.48	4.72
216	1817.59	4.61
243	1818.31	5.67
280	1818.90	5.76
310	1819.47	5.17
340	1820.28	3.67
400	1820.58	4.81
460	1822.21	5.14
520	1823.30	4.12

* Coincident with tributary

5.3 Channel Gradation

As noted in Section 3.2, the subsurface investigation and testing conducted by Shawal GMTL is summarized in a report dated 29 May 2014. This report includes gradation logs at the two test pits performed in the channel. Based on subsequent conversations with Shawal GMTL, the “1.0 m” gradation logs are actually composite logs based on the samples they performed in depths from 0.0 to 3.0 meters. The gradation tests were based on a maximum sieve size of 75 mm (3 inches). Particles greater than 75 mm in diameter were weighed and accounted for in the reported gradations.

A summary of the d_{50} values for the test pits is presented in Table 3. The results of the gradation analyses show that minimum d_{50} for the test pit samples is approximately 5.7 mm and was used for the scour analysis. Values for d_{50} on the boring samples at all depths were not considered in this analysis. The method of obtaining the samples from depth makes it physically impossible to obtain particle sizes greater than 50 mm, which is not representative of the riverbed soil. Without the larger particle sizes in the sample, gradation results will be biased to the smaller particle sizes and will report a smaller d_{50} than normal. The smaller values were not considered to be representative of the overall stream system and the values were neglected for scour analysis. The selected d_{50} for the analysis was 5.7 mm. This value was selected because it was considered to be the smallest d_{50} for the soils that would normally aggrade or degrade during flood events.

Table 3
Summary of d_{50} (mm) for test pits

Test Pit ID	d_{50} (mm)
TP-1	5.7
TP-2	8.0

5.4 Scour Analysis

Scour potential at structures is a combination of long term scour, contraction scour, and localized scour at the abutments piers. Long term aggradation or degradation is the raising or lowering of the stream bed due to natural stream formation processes. Contraction scour can occur when flow is constricted from a wider floodplain into a narrower area, such as a bridge, and can occur over the entire streambed. Localized scour at abutments and piers is typically a result of vortices in flow. Localized scour is added to the contraction scour and long term scour. Contraction and localized scour analysis was performed using the HEC-RAS program.

Long term aggradation or degradation of the streambed may be considered in a scour analysis, but requires significant monitoring and analysis of the streambed over time in order to develop an estimate of long term aggradation and/or degradation. No data for this river was available for review, thus long term aggradation and/or degradation are not accounted for numerically in this analysis. Further, the potential for deposition or high sediment loading under high flow conditions is unknown and thus not considered in the overall hydraulic design-based recommendations. As previously noted, photographs in the survey report depict the river and tributary as braided, and cobble dominated systems with high width to depth ratios. It is possible these systems have high sediment supply, with the potential for excessive deposition both longitudinally and transversely. The banks appear to be erosive, likely a result of lateral movement of the river in response to significant flows. These observations lead to two recommendations: 1) provide bank stabilization in the vicinity of the bridge to stabilize the channel approaches, and 2) implement a monitoring program for changes in channel bed, including deposition, and perform maintenance to maintain the design dimension and elevations.

Contraction scour can either be clear water scour or live bed scour. Clear water scour can occur when the sediment in the uncontracted approach section is less than the sediment carrying capacity for that flow. Because this river is in a natural state, i.e. there are no dams or other factors to reduce sediment within the creek, and because it has high velocities, clear water scour was considered to be unlikely. Live bed scour, where some sediment load is carried into the crossing, was used for this analysis. This assumption is verified in HEC-RAS by the comparison of critical velocity, the velocity required to move the average size material, with the computed velocities. Calculations indicate the computed velocities exceed the critical values, thus supporting the live bed scour approach to this analysis.

Methods, equations, and coefficients for scour calculations are detailed in the HEC-RAS *Hydraulic Reference Manual* and HEC-18 *Estimating Scour at Bridges*. HEC-RAS utilizes Laursen's live-bed contraction scour analysis. Pier scour and abutment scour can be calculated using one of several methods available in HEC-RAS. The Colorado State University (CSU) equation was selected for estimating pier scour and the Froehlich Equation was selected for estimating abutment scour. No wood debris accumulation was considered in the pier width based on the lack of timber observed in the photos.

A summary of the calculated scour results is presented in Table 4. The values for the top of footing of the abutments summarized in the table below was calculated as the minimum channel elevation, located at the downstream end of the bridge, minus the scour depth. The maximum top of footing elevation for the pier was estimated by the model and differs from the modeled result included in the appendices. The scour depth from the model is estimated using the equations in HEC-18. However, the scour cavities from the abutments are larger than the modeled pier scour depth. The modeled abutment scour cavities have sufficient depth that the cavity is larger than the pier scour cavity. In addition, the material remaining under the pier is expected to be insufficient for structural support. It is recommended to establish the top of pier elevation as the same elevation for the abutments.

Table 4
2010 Design Scour Depths

	West Abutment (left)	**Pier**	**East Abutment (right)**
Total scour depth	10.35 m	1.14 m	10.35 m
Minimum channel elevation (downstream side of bridge)	1815.51		
Maximum top of footing elevation for scour protection	1805.15 m	1805.15 m	1805.15 m

Generally, if the flow velocity in the stream is less than the threshold flow velocity for mobilization of bed material, a riprap blanket around the pier might help reduce scour. However, in the case of Bridge #10, the channel velocities are greater than that required for mobilization so the use of riprap at the piers is discouraged because the loose riprap will break up (dissipate) due to the secondary flow patterns at and around the piers, and sink down into the streambed offering no protection from scour at the piers.

5.5 Recommended Design Parameters

Based on the 2010 Design, the site investigations and the hydraulic modeling, Tetra Tech recommends that the Structural Evaluation (see Section 6.0) for Bridge #10 be based on the following design parameters:

- River bed elevation of 1816.370m at the upstream face of the bridge
- River bed elevation of 1815.510 m at the downstream face of the bridge
- Verifications that pier and abutment footings are below the scour line
- Verification that the proposed bridge seat elevation has been set a minimum of 600mm above the 50-year flood elevation.
- Hydraulic Data:
    - Design Flood Event = 50-yr
    - Design Velocity = 4.34 m/s
    - Design Water Surface Elevation =1819.17m

- Scour Consideration at the Abutments:
    - Scour Depth = 10.35m
    - Max. Top of Footing Elevation = 1805.15m
- Scour Considerations at the Pier:
    - Scour Depth = 1.14m
    - Max. Top of Footing Elevation = 1805.15m (governed by the deep scour cavities at the abutments)

6.0 Structural Evaluation

6.1 General

Based on the 2010 Design, the proposed bridge is a two-span cast-in-place slab bridge comprised of 12.17 meter simple spans, with a total bridge length of 24.34 meters. The superstructure (cast-in-place slab and barriers) and the substructure (abutments, retaining walls and pier) are reinforced concrete. The roadway has two 4.0m wide travel lanes and has two 1.2 m wide sidewalks on each side of the roadway. The AASHTO design vehicle used in the 2010 Design is unknown.

The purpose of Tetra Tech's structural analysis is to determine if the 2010 Design for Bridge #10 is adequate based on the following criteria:

- Hydraulics - Adequate hydraulic opening and scour protection
- Geotechnical - Adequate for stability (ultimate bearing resistance of the soil, overturning, sliding)
- AASHTO LRFD requirements (per AASHTO LRFD Bridge Design Specifications, 6th Edition, 2012)
- Constructability
- Safety
- Maintenance

6.2 Load Distribution

Based on the bridge structure as detailed in the 2010 Design plans, Tetra Tech performed calculations in order to determine the adequacy of the substructure. In order to do this, Tetra Tech distributed the loads from the superstructure (slab, barriers, and sidewalk) to each substructure element. Since the 2010 Design included a dowel between the superstructure and each substructure element near the bearings, in addition to the vertical superstructure loads being transferred to each abutment and pier, the lateral superstructure loads were also transferred to each abutment and pier.

The significance of this load distribution is that it is not typical. In a typical bridge, elastomeric bearings are used to allow thermal expansion in some locations and to fix movement in other locations (typically with the use of anchor bolts). Due to the use of the dowel between the superstructure and substructure, the 2010 Design does not permit any expansion or freedom of movement between the superstructure and substructure.

In addition to the superstructure reactions, loads due to lateral earth pressure, seismic and stream flow were applied to the abutments and pier in accordance with AASHTO LFRD load combinations.

The primary load cases considered in the analysis are as follows:

- Dead Load: Selfweight of superstructure and substructure components
- Live Load: AASHTO LRFD HL-93 Vehicle
- Longitudinal Force: 5% of Live Load
- Seismic Load: Ss = 0.64g S1 = 0.47g
SDC D PGA = 0.29g
(See the calculations in Appendix B for back-up)
- Hydraulic Data: As noted under Section 5.5
- Geotech Data: As noted under Section 4.2

6.3 Hydraulic Evaluation

As discussed in Section 5.5, the bridge crossing was evaluated for an upstream river bed elevation of 1816.37 m and a 50-year flood elevation of 1819.17 m. Using the bottom of beam elevation in the 2010 Design (El. 1819.9m), the calculated freeboard is approximately 700mm, which is greater than the generally observed minimum freeboard of 600 mm. Based on the 2010 Design, the clear span between abutments is 22.94 m and the elevation of the low point of the superstructure is 1819.90 m. Therefore, the 2010 Design provides a hydraulic opening of approximately 77.45 m^2, which accounts for some obstructed area due to the pier. The bridge and assumed channelization were encoded into the hydraulic model for evaluation. In conjunction with the assumed channelization, the bridge opening has sufficient capacity for the 50-year peak discharge.

Similarly, a comparison between the 2010 Design top of footing elevations and the required top of footing elevations required for scour protection (see Section 5.4) is as follows:

Table 5
2010 Structural Elevations and Scour Depths

	2010 Design Top of Footing El. (m)	Tetra Tech Computed Max Top of Footing Elevation for Scour Protection (m)
North Abutment	1812.10	1805.15
Pier	1813.50	1805.15
South Abutment	1812.10	1805.15

<u>**Based on these values, the 2010 Design is not adequate for scour.**</u>

6.4 Geotechnical Evaluation

Tetra Tech analyzed the Bridge #10 abutments and piers to determine the ultimate bearing resistance required for stability in accordance with AASHTO LRFD. These calculations can be found in “Engineering Support Program, WO-LT0077, Gardez to Khost Road, Bridge #10, Geotechnical Report” dated June 17, 2014.

Since the hydraulic analysis determined that the 2010 Design was not adequate for scour, the structural stability analysis was based on the incorporation of a concrete scour pad under the bridge (see Section 7.2), and thus full passive pressure / resistance was included in the bridge analysis. It should be noted that passive pressure / resistance is typically neglected in substructure design since the soil in front of the abutments/pier is subject to scour.

A comparison between the bearing resistance required for the 2010 Design and the Ultimate Bearing Resistance (the capacity available) is as follows:

Case 1 – Non-Seismic (Dead Load, Live Load, Earth Pressure):

	Bearing Resistance Required [kN/m^2(ksf)]	**Ultimate Bearing Resistance(Capacity) [kN/m^2(ksf)]**	**Conclusion**
Abutments	245 (5.11)	381 (8.00)	OK
Pier	275 (5.75)	561 (11.75)	OK

Case 2 – Seismic (Dead Load, Seismic):

	Bearing Resistance Required [kN/m^2(ksf)]	**Ultimate Bearing Resistance(Capacity) [kN/m^2(ksf)]**	**Conclusion**
Abutments	Not able to be computed **	847 (17.78)	No Good
Pier	615 (12.84)	1246 (26.00)	OK

** Due to the large eccentricity of the controlling load combination, the AASHTO formulas for bearing resistance result in negative values since the eccentricity is outside the acceptable range.

In addition to bearing resistance, resistance against Overturning and Sliding were checked. The 2010 Design did not meet AASHTO LRFD requirements for:

- Overturning or Sliding for the abutments under seismic loading
- Sliding for the pier under seismic loading.

<u>Based on these values, the 2010 Design Bridge #10 is not adequate for overall stability (bearing resistance, overturning, sliding) for seismic load conditions.</u>

6.5 Additional Structural Concerns

Analyzing the strength of the reinforced concrete superstructure and substructure was not part of this assignment. However, based on the 2010 Design not meeting stability requirements per AASHTO LRFD, it should be anticipated that the 2010 Design may not meet AASHTO LRFD strength requirements either. Based on the results above, an area of particular concern would be the flexural capacity of the base of the abutment stem and in the footing.

The only element providing restraint of the superstructure are the steel dowels (discussed in Section 6.2), which result in a lack of relative movement between the superstructure and substructure and should be anticipated to cause cracking and result in increased maintenance. It should be noted that the 2010 Design does not include cheekwalls at the outside of the abutments or the pier to restrain the superstructure during a seismic event. This is a Life Safety concern - when the doweled connections fail, there is nothing to restrain the superstructure.

7.0 Recommendations & Related Costs

7.1 3-Span Alternate

Since Tetra Tech recently completed the 3-span design of Bridge #09 which was based on similar subsurface conditions and scour concerns to those at Bridge #10, Tetra Tech ran a subsequent hydraulic model based on geometry similar to the 3-span Bridge #09 structure. The resulting hydraulic analysis reflected a decrease in scour depth of approximately 2 m from the 2010 Design configuration. Adding a third span to the project would introduce costs associated with an additional span and an additional pier, as well as costs associated with realigning and widening the channel. Footing depths are still very significant for this alternative and present the same constructability concerns as the 2010 design. Since the scour depths are not significantly decreased, Tetra Tech feels that the additional costs associated with this alternate are not justified, and there **a 3-span Alternate is not recommended.**

7.2 Scour Protection Alternate

Several alternatives were considered for protection of the piers and abutments from the calculated scour depths. Alternatives that were evaluated include deeper spread footing foundations, drilled foundations, concrete armoring of the channel, and armoring the channel with articulated concrete blocks. Evaluations included constructability, cost, availability of skilled labor and equipment and schedule. Similar to our experience with Bridge #09, **a concrete apron is recommended to armor the channel.** The concrete apron should include downward sloping key walls to protect the apron from undermining.

The concrete apron is intended to prevent the formation of scour holes at the pier and abutment. By covering the riverbed soil, scour holes are not able to propagate out from the structure where they form. Some local scour is anticipated at the edges of the apron where flow transitions back to normal river flows. No research has been done for this specific type of application. An estimate for this local scour was adapted from existing methods to determine the approximate depth.

A calculation for general scour using *Technical Supplement 14B* of the National Engineering Handbook was used to estimate general scour depth. The general river scour estimate is noted as equation TS14B-23 in the publication. The equation for general scour is:

$$z_t = KQ_d^aW_f^bd_{50}^c$$

Where:

z_t	maximum scour depth (m)
K	coefficient from table TS14B-8
Q_d	design discharge (m^3/s)
W_f	flow width (m)
d_{50}	median size of bed material (mm)
a,b,c	exponents from table TS14B-8

Coefficients and exponents in the equation are determined by the general geometry of the river. In this location, the "right angle" coefficients and exponents were selected because the river does turn approximately 90 degrees just downstream of the bridge. Coefficients also vary based on experimental data by two researchers (Lacey and Blench). For the purposes of this evaluation, both data sets are utilized for calculations. The d_{50} of the material used for this calculation was approximately 5.7 mm, which is the average d_{50} determined from laboratory data.

Using the selected parameters above and data from the HEC-RAS model, the estimated scour depth using the Lacey relations was approximately 1.7 meters. The estimated scour depth using the Blench relations is approximately 3.0 meters.

The calculated scour depths show satisfactory correspondence between the two methods. To provide a factor of safety, the sloped key walls for the apron are recommended to be set to a depth of 3.5 meters below the edge of apron.

Tetra Tech evaluated the potential for uplift of the concrete mat at varying flow conditions across the mat. Velocities for each flow condition were used to determine the uplift force that the mat would experience. Forces that were calculated to counteract the uplift forces were the weight of the mat itself and the weight of the water above the concrete mat. The typical factor of safety used for uplift resistance is 1.5.

The nominal mat thickness used in the analysis was 0.20 meters (8 inches). Calculations for uplift for the apron were based on the assumption that the channel would be graded as described in preceding sections of this report. Results of the uplift calculations show that this apron thickness should be sufficient to resist uplift forces. Additional calculations would be needed if this apron is developed as a design alternative.

Tetra Tech recommends construction of the scour protection apron in conjunction with any of the recommendations. If a scour protection apron is not selected, alternative foundation designs would need to be prepared to account for the scour depth.

7.3 Raised Roadway Alternate

Previous sections discussing roadway and bridge elevations are based on the 2010 Design Bridge plans. It should be noted that the 2010 Design Bridge plans showed a top of roadway elevation on the bridge of Elev. 1820.614 (per Volume 3 / Bridgework package dated March 2010), whereas the 2010 Design Roadway plans showed a top of roadway elevation on the bridge of Elev. 1820.050 (per Volume 2 / Roadwork package dated June 2010). In both drawing volumes, the top of roadway has a cross-slope but the roadway profile elevation is consistent (flat) across the bridge.

The roadway profile in the 2010 Design consists of a 5% slope down to the bridge on the north side and a 2% slope down to the bridge on the south side. Since the bridge is flat and there are concrete barriers on both sides of the roadway along the bridge and approaches (approximately 35 meters in length total), stormwater from the roadway upstation and downstation of the bridge will flow toward the low point of the profile (the bridge) and be trapped on the bridge. To address stormwater, the 2010 Design includes scuppers. However, without regular maintenance, scuppers typically become clogged and ineffective. It should also be noted that poor bridge drainage will lead to increased deterioration and required maintenance of bridge components. Therefore, from a drainage standpoint, the following profile recommendations are recommended:

- North of the Bridge - Adjust the profile to redirect / minimize flow on the bridge.

- On the bridge – Adjust the profile to promote flow across the bridge and reduce dependency on working scuppers.
- South of the Bridge – Adjust the profile to redirect / minimize flow on the bridge.
- Coordination of elevations on Bridge and Roadway plans.

In addition to the stormwater concern, there is a concern that the roadway will overtop at the bridge during a flood event due to the low elevation of the agricultural fields on the north bank of the tributary. The fields can trap flows at a higher water surface elevation than expected because they would be continuously fed from upstream flooding sections and unable to drain through the bridge. Flooding in the fields could potentially overtop the north approach to the bridge. This situation could be remedied by increasing the approach road elevation or hydraulically connecting the east side of the fields to the tributary and bridge.

7.4 Modified Structure Alternate

As discussed in Section 6.0, Tetra Tech recommends redesign of the Bridge #10 superstructure and substructure. Since the geotechnical and hydraulic conditions at Bridge #09 are similar to those at Bridge #10, **Tetra Tech recommends redesigning the 2-span Bridge #10 based on the Bridge #09 design**. This will translate into both an economy of design costs and also design duration.

The substructure redesign will include wider footings and will incorporate cheekwalls on the abutments and piers to restrain the superstructure laterally during a seismic event. The superstructure redesign will consist of a concrete slab / concrete beam system, supported on elastomeric bearings. A combination of fixed and expansion bearings will be used to promote relative movement between the superstructure and the substructure. The proposed design will be AASHTO LRFD compliant.

Hydraulic analysis shows that this design has sufficient capacity to convey the 50-year flood. Scour analysis results also show that the scour depth for this configuration is approximately 9.30 meters at the abutments and approximately 1.08 meters at the pier. It is recommended that the pier footing be set at the same elevation of the abutment footings due to the instability of the remaining soil "pillar" in the channel. This is due to the size of the abutment scour cavities. The scour protection apron is recommended to be constructed in conjunction with this alternate.

7.5 Alternate Costs

Tetra Tech developed order-of-magnitude cost estimate calculations in order to determine the impact of these recommendations. These calculations are included in Appendix C.

The Modified Structure Alternate discussed in Section 7.4 will result in longer spans and therefore larger structure costs. The cost differential to the project will be the difference in quantities from the proposed bridge and the 2010 Design, including additional concrete and reinforcement in the superstructure and substructure, plus the addition of elastomeric bearings and retaining walls into the proposed design. Tetra Tech estimates that the Modified Structure Alternate would increase the project cost by approximately $190,000.

The Scour Protection Alternate discussed in Section 7.2 will involve construction of a 200mm thick concrete slab with sloping cut-off walls along the upstream and downstream edges. The limit of the concrete slab will extend approximately 14m upstream and downstream of the bridge (10.5m along the channel bottom and 3.5 m sloping cut-off wall). The cost of the concrete scour mattress would depend on whether the Modified Structure Alternate was adopted, since that alternate has a longer total structure length than the 2010 Design. Tetra Tech estimates that adding a concrete scour mattress to the 2010 Design would add $105,000 to the project, and adding the concrete scour mattress to the Modified Structure Alternate would add $155,000 to the project.

The Raised Roadway Alternate discussed in Section 7.3 will involve raising the roadway profile to match the bridge drawings, hydraulically connecting the east side of the fields to the tributary and bridge, updating the roadway profile north and south of the bridge as well as on the bridge to promote drainage off the bridge, and updating the substructure heights to reflect the updated profile. This alternate involves earthwork, roadway work, drainage work and modifications to the abutment and pier heights. Tetra Tech estimates that the Raised Roadway Alternate will add $40,000 to the project.

Not included in these figures are the cost benefits associated with reduced long-term maintenance and having a bridge crossing which has been designed in accordance with AASHTO LRFD to withstand a seismic event and a 50-year flood event.

8.0 Summary & Next Steps

Based on the results of the hydraulic analyses, the 2010 Design is adequate for hydraulic capacity and freeboard clearance above the 50-year flood elevation, but is not adequate for scour. Scour depths at the abutments are significant (approximate 10 meters) and would result in not only undermining of the abutments, but also undermining of the pier. Due to the depth of the scour holes and the velocity of the flow in the channel, armoring the channel with riprap is not sufficient. Tetra Tech recommends using a cast-in-place reinforced concrete mattress to protect the bridge substructure from scour.

Based on the results of the geotechnical and structural analyses, the 2010 Design does not meet AASHTO LRFD code requirements for overall stability (bearing resistance, overturning and sliding) of the abutments and pier during a seismic event. In addition, the 2010 Design uses steel dowels for lateral restraint during a seismic event which also prevent relative movement between the superstructure and substructure even due to thermal loads. Tetra Tech recommends using fixed/expansion elastomeric bearings to allow relative movement, and recommends adding cheekwalls to the substructure for seismic restraint. Since both changes in the superstructure and substructure design are recommended, Tetra Tech recommends that the new 2-span Bridge #10 design be similar to the recently designed Bridge #9 in order to economize design costs and schedule, and streamline construction of the two bridges by using similar procedures, forms and materials.

Tetra Tech also recommends modifying the roadway profile such that the low point is moved off the bridge, a slope is provided along the bridge and the northern and southern approaches are modified to promote drainage off the bridge. In comparison to the 2010 Design Roadway plans which was not coordinated with the 2010 Design Bridge plans, the resulting profile will be raised.

Tetra Tech also recommends channel improvements to reduce flooding concerns, including demolition of the existing causeway and hydraulically connecting the east side of the fields north of the tributary to the tributary and the bridge.

If all the recommendations are incorporated, Tetra Tech estimates that it will increase the Bridge #10 project cost approximately $385,000 over the 2010 Design construction cost. This is an order-of-magnitude cost estimate to be used for budget purposes only. Tetra Tech feels these recommendations are warranted since the 2010 Design does not meet AASHTO LRFD code requirements, does not provide adequate protection of life safety in a seismic event, does not provide adequate protection against flooding and does not provide adequate protection to withstand scour.

No additional field information would be required in order to perform the final design for Bridge #10 based on these recommendations. The final design effort would consist of highway and bridge design work, along with limited geotechnical and hydraulic work as needed to support the final bridge design. The redesign would consist of developing a set of civil and structural drawings, and calculations required to support them. A Design Analysis will be prepared summarizing the Bridge #10 design information. Technical Specifications will also be submitted. Due to the discrepancy between the 2010 Design Bridge and Roadway plans, Tetra Tech would require confirmation of the proposed roadway profile at the tie-in locations north and south of the bridge prior to commencing final design.

Appendix A

Hydraulic Modeling

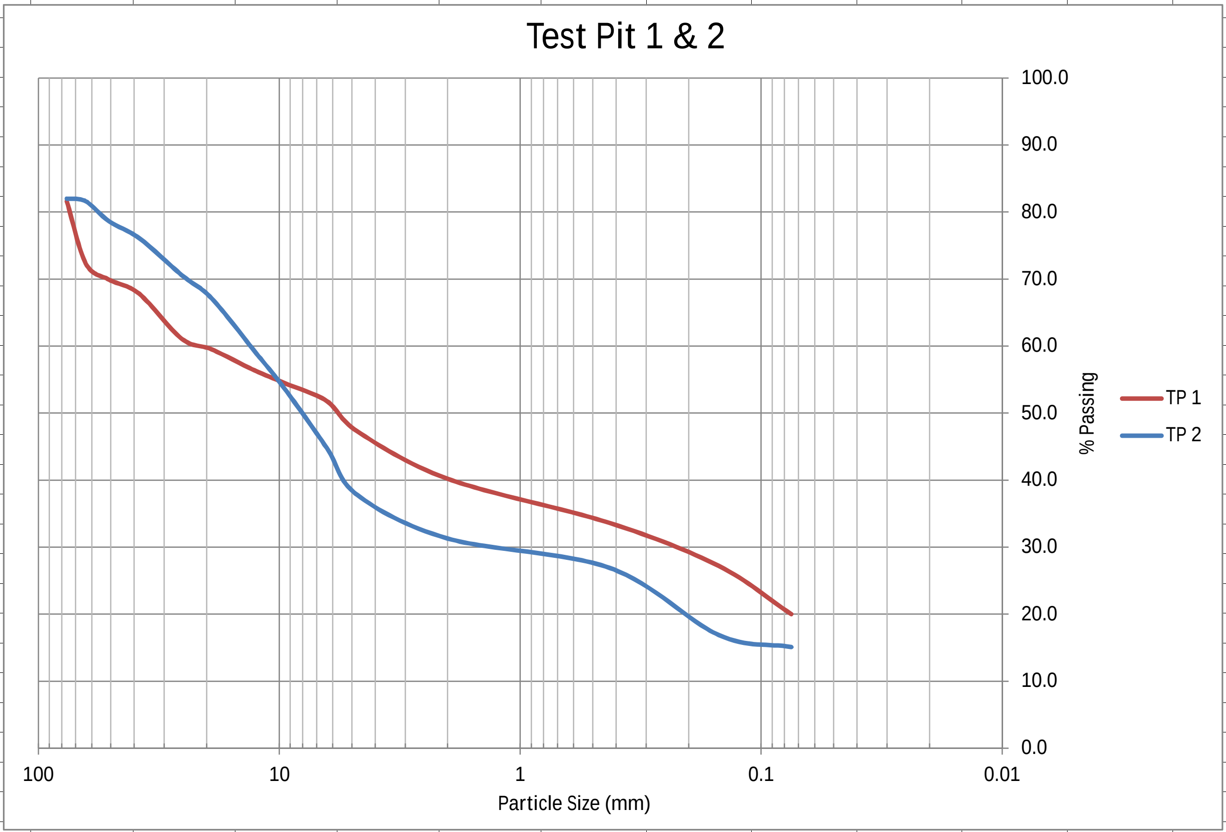
Gardez-Khost Road Bridge #10

SAMPLE TP 01 and 02 - Composite Sample (all depths)

Percent Passing at Specified Depth			
Sieve Name	Sieve Size (mm)	TP - 1	TP -2
3"	76.2	81.6	82.0
2.5"	63.5	72.3	81.6
2"	50.8	69.9	78.6
1.5"	38.1	67.8	76.1
1"	25.4	61.1	70.6
3/4"	19.05	59.5	67.1
1/2"	12.7	56.4	59.3
3/8"	9.53	54.5	53.7
1/4"	6.35	51.8	44.8
No. 4	4.75	47.3	37.8
No. 10	2	40.2	31.3
No. 40	0.425	33.6	26.9
No. 100	0.150	27.2	16.9
No. 200	0.075	20.0	15.1

D50 for each depth (mm)

Test Pit	D50 (mm)
TP 1	5.7
TP 2	8



Geotechnical data summarized from Geotechnical Report Addendum by Shawal Geotechnical Laboratory

Gardez-Khost Road Bridge #10
HEC-RAS Results
Proposed Tributary Hydraulic Model

Model Features:
2-span bridge per 2010 Design
Channel graded to bridge approach

Profile 1 - Assumes no flooding in Main River
Profile 2 - Assumes coinicident peak flooding in Main River

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)	
Tributary	20	PF 1	185.3	1813.26	1815.17	1815.17	1815.66	0.020866	3.11	59.53	62	1.01
Tributary	20	PF 2	185.3	1813.26	1816.84	1815.17	1816.89	0.000708	1.02	180.86	78.03	0.21
Tributary	33	PF 1	185.3	1813.77	1815.48		1815.86	0.011431	2.86	71.19	67.76	0.79
Tributary	33	PF 2	185.3	1813.77	1817.47		1817.5	0.000343	0.91	232.67	86.68	0.16
Tributary	48	PF 1	185.3	1813.99	1815.66	1815.43	1816.07	0.015392	3.06	67.42	70.29	0.9
Tributary	48	PF 2	185.3	1813.99	1817.48		1817.52	0.000469	0.92	210.59	84.64	0.18
Tributary	62	PF 1	185.3	1814.41	1815.95		1816.26	0.010758	2.61	75.69	71.32	0.76
Tributary	62	PF 2	185.3	1814.41	1817.59		1817.63	0.000505	1	202.37	81.95	0.19
Tributary	95	PF 1	185.3	1815.51	1817.23	1817.23	1818.04	0.017879	3.97	46.67	28.79	1
Tributary	95	PF 2	185.3	1815.51	1817.23	1817.23	1818.04	0.017879	3.97	46.67	28.79	1
Tributary	100		Bridge									
Tributary	113	PF 1	185.3	1816.31	1819.17	1818.03	1819.44	0.003215	2.3	80.59	31.06	0.45
Tributary	113	PF 2	185.3	1816.31	1819.17	1818.03	1819.44	0.003215	2.3	80.59	31.06	0.45
Tributary	134	PF 1	185.3	1817.24	1819.35	1819.35	1820.36	0.018638	4.45	41.65	22.8	0.99
Tributary	134	PF 2	185.3	1817.24	1819.35	1819.35	1820.36	0.018638	4.45	41.65	22.8	0.99
Tributary	167	PF 1	185.3	1818.74	1821.3	1821.3	1822.01	0.01254	4.26	59.07	95.97	0.89
Tributary	167	PF 2	185.3	1818.74	1821.3	1821.3	1822.01	0.01254	4.26	59.07	95.97	0.89
Tributary	194	PF 1	185.3	1819.5	1822.07		1822.31	0.007017	3.14	104.46	88.34	0.67
Tributary	194	PF 2	185.3	1819.5	1822.07		1822.31	0.007017	3.14	104.46	88.34	0.67
Tributary	225	PF 1	185.3	1820.26	1822.23	1822.23	1822.72	0.017274	4.19	71.93	70.21	1.01
Tributary	225	PF 2	185.3	1820.26	1822.23	1822.23	1822.72	0.017274	4.19	71.93	70.21	1.01
Tributary	254	PF 1	185.3	1820.59	1823.1	1823.1	1823.52	0.010362	3.67	83.87	85.75	0.81
Tributary	254	PF 2	185.3	1820.59	1823.1	1823.1	1823.52	0.010362	3.67	83.87	85.75	0.81

Gardez-Khost Road Bridge #10													
HEC-RAS Results													
Main River Hydraulic Model													
Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl	
			(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)		
Main River	20	PF 1	820	1811.26	1814.32	1814.02	1815.11	0.010013	4.48	211.02	91.59	0.85	
Main River	80	PF 1	820	1811.9	1815.16	1815.16	1816.18	0.013851	5.34	188.89	90.86	1	
Main River	140	PF 1	820	1812.7	1816.24	1816.24	1817.39	0.013828	5.91	177.63	76.17	1.02	
Main River	170	PF 1	820	1813.26	1816.84	1816.84	1817.95	0.013389	5.82	180.77	78.03	1.01	
Main River	185	PF 1	820	1813.64	1817.47		1818.14	0.006624	4.22	232.34	86.67	0.71	
Main River	200	PF 1	820	1813.99	1817.48		1818.29	0.00895	4.72	210.64	84.65	0.82	
Main River	216	PF 1	820	1814.34	1817.59		1818.45	0.009879	4.61	202.03	81.93	0.85	
Main River	243	PF 1	820	1814.79	1818.31	1818.31	1819.54	0.013486	5.67	172.22	69.61	0.99	
Main River	280	PF 1	820	1815.24	1818.9	1818.9	1820.2	0.013697	5.76	165.85	63.21	1	
Main River	310	PF 1	820	1815.78	1819.47		1820.58	0.010785	5.17	178.9	64.62	0.9	
Main River	340	PF 1	820	1816.38	1820.28		1820.85	0.005135	3.67	248.91	82.15	0.62	
Main River	400	PF 1	820	1817.54	1820.58	1820.58	1821.57	0.013575	4.81	192.61	96.82	0.96	
Main River	460	PF 1	820	1818.56	1822.21	1822.21	1823.31	0.011949	5.14	188.18	86.14	0.93	
Main River	520	PF 1	820	1819.45	1823.3		1823.86	0.005914	4.12	254.29	99.74	0.68	

Gardez-Khost Road Bridge #10

HEC-RAS Results

Proposed Tributary Hydraulic Model

Model Features:

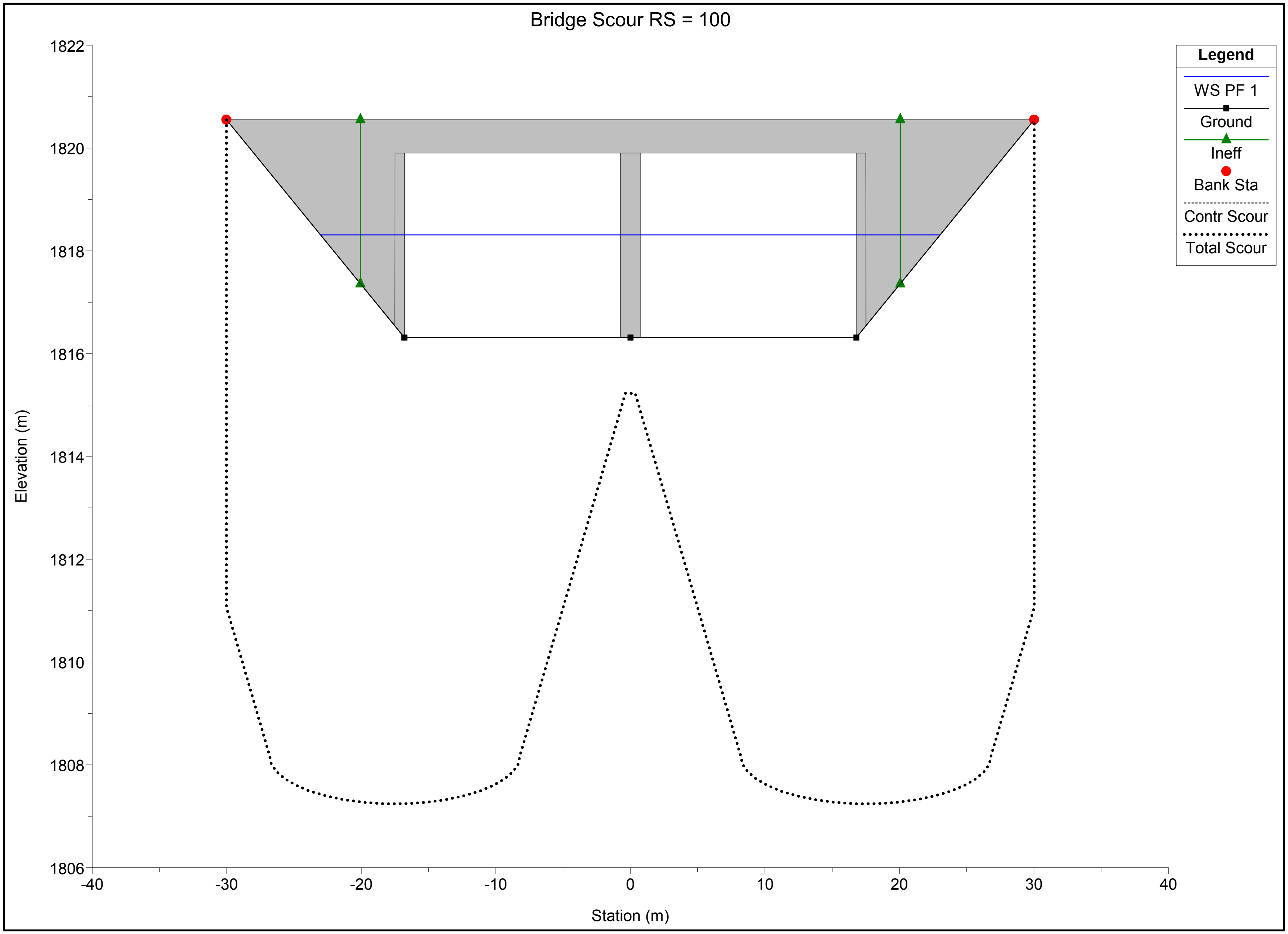
2-span bridge, adapted from Bridge 9 Design

Channel graded to bridge approach

Profile 1 - Assumes no flooding in Main River

Profile 2 - Assumes coinicident peak flooding in Main River

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)	
Tributary	20	PF 1	185.3	1813.26	1815.17	1815.17	1815.66	0.020866	3.11	59.53	62	1.01
Tributary	20	PF 2	185.3	1813.26	1816.84	1815.17	1816.89	0.000708	1.02	180.86	78.03	0.21
Tributary	33	PF 1	185.3	1813.77	1815.48		1815.86	0.011431	2.86	71.19	67.76	0.79
Tributary	33	PF 2	185.3	1813.77	1817.47		1817.5	0.000343	0.91	232.67	86.68	0.16
Tributary	48	PF 1	185.3	1813.99	1815.66	1815.43	1816.07	0.015392	3.06	67.42	70.29	0.9
Tributary	48	PF 2	185.3	1813.99	1817.48		1817.52	0.000469	0.92	210.59	84.64	0.18
Tributary	62	PF 1	185.3	1814.41	1815.95		1816.26	0.010758	2.61	75.69	71.32	0.76
Tributary	62	PF 2	185.3	1814.41	1817.59		1817.63	0.000505	1	202.37	81.95	0.19
Tributary	95	PF 1	185.3	1815.51	1816.9	1816.9	1817.53	0.018169	3.51	52.77	42.29	0.99
Tributary	95	PF 2	185.3	1815.51	1817.44		1817.76	0.005611	2.47	75.07	45.66	0.58
Tributary	100		Bridge									
Tributary	113	PF 1	185.3	1816.31	1818.52	1817.69	1818.76	0.003565	2.18	85.07	47.35	0.48
Tributary	113	PF 2	185.3	1816.31	1818.36	1817.69	1818.64	0.004587	2.35	78.88	46.39	0.53
Tributary	134	PF 1	185.3	1817.24	1819.04	1819.04	1819.91	0.018421	4.12	44.98	25.58	0.99
Tributary	134	PF 2	185.3	1817.24	1819.04	1819.04	1819.91	0.018421	4.12	44.98	25.58	0.99
Tributary	167	PF 1	185.3	1818.74	1821.3	1821.3	1822.01	0.01254	4.26	59.07	95.97	0.89
Tributary	167	PF 2	185.3	1818.74	1821.3	1821.3	1822.01	0.01254	4.26	59.07	95.97	0.89
Tributary	194	PF 1	185.3	1819.5	1822.07		1822.31	0.007017	3.14	104.46	88.34	0.67
Tributary	194	PF 2	185.3	1819.5	1822.07		1822.31	0.007017	3.14	104.46	88.34	0.67
Tributary	225	PF 1	185.3	1820.26	1822.23	1822.23	1822.72	0.017274	4.19	71.93	70.21	1.01
Tributary	225	PF 2	185.3	1820.26	1822.23	1822.23	1822.72	0.017274	4.19	71.93	70.21	1.01
Tributary	254	PF 1	185.3	1820.59	1823.1	1823.1	1823.52	0.010362	3.67	83.87	85.75	0.81
Tributary	254	PF 2	185.3	1820.59	1823.1	1823.1	1823.52	0.010362	3.67	83.87	85.75	0.81


Bridge Scour RS = 100
Legend
WS PF 1
Ground
Ineff
Bank Sta
Contr Scour
Total Scour
Elevation (m)
Station (m)
1822
1820
1818
1816
1814
1812
1810
1808
1806
-40
-30
-20
-10
0
10
20
30
40

Contraction Scour

		Left	Channel	Right
Input Data				
	Average Depth (m):		2.04	
	Approach Velocity (m/s):		4.45	
	Br Average Depth (m):		1.94	
	BR Opening Flow (m3/s):		185.30	
	BR Top WD (m):		21.96	
	Grain Size D50 (mm):		5.70	
	Approach Flow (m3/s):		185.30	
	Approach Top WD (m):		20.41	
	K1 Coefficient:		0.640	
Results				
	Scour Depth Ys (m):		0.01	
	Critical Velocity (m/s):		1.24	
	Equation:		Live	

Pier Scour

All piers have the same scour depth

Input Data		
	Pier Shape:	Round nose
	Pier Width (m):	1.50
	Grain Size D50 (mm):	5.70000
	Depth Upstream (m):	2.66
	Velocity Upstream (m/s):	2.28
	K1 Nose Shape:	1.00
	Pier Angle:	0.00
	Pier Length (m):	10.95
	K2 Angle Coef:	1.00
	K3 Bed Cond Coef:	1.10
	Grain Size D90 (mm):	100.00000
	K4 Armouring Coef:	0.40
Results		
	Scour Depth Ys (m):	1.14
	Froude #:	0.45
	Equation:	CSU equation

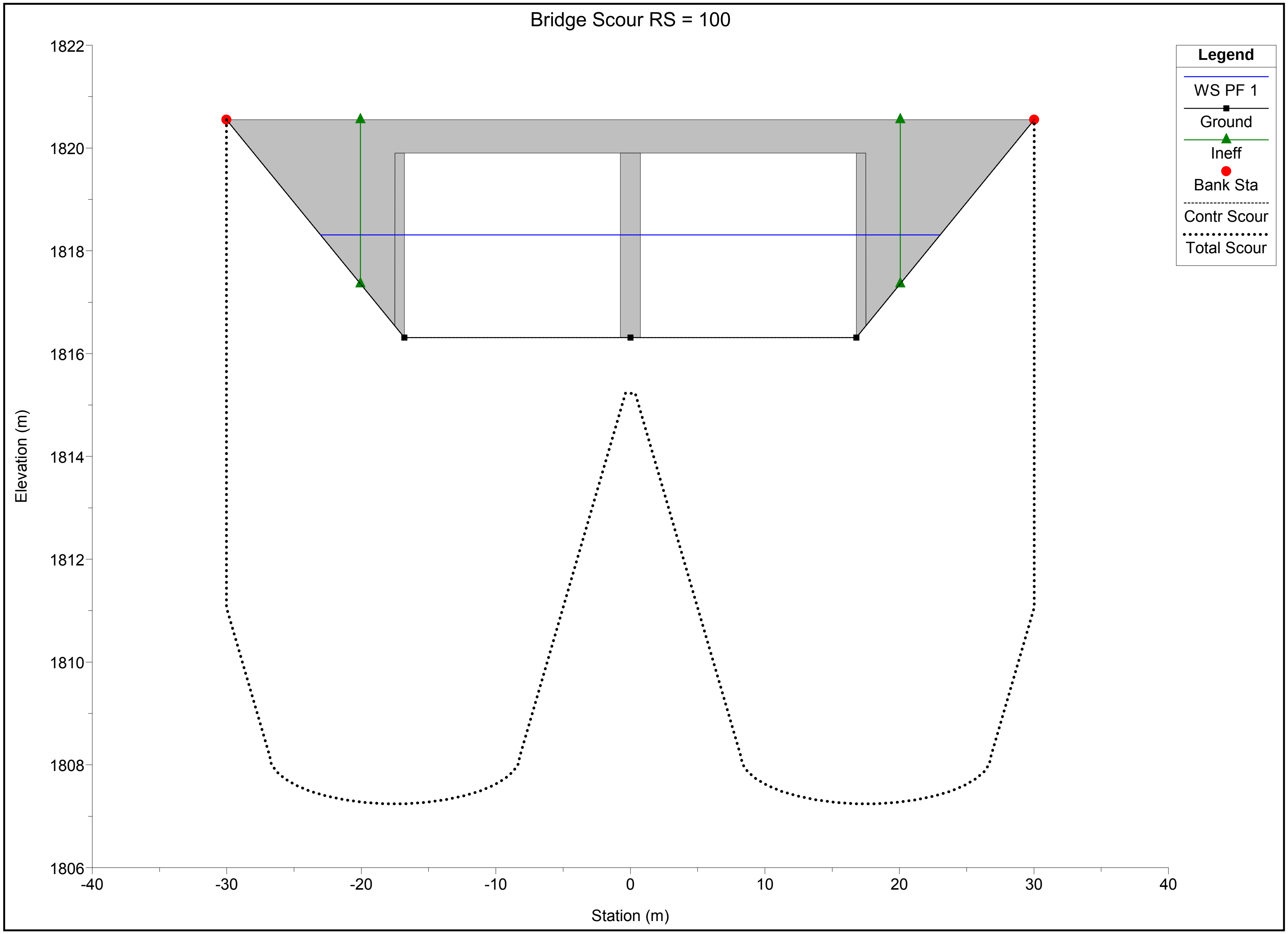
Abutment Scour

		Left	Right
Input Data			
	Station at Toe (m):	-12.69	12.69
	Toe Sta at appr (m):	96.30	91.33
	Abutment Length (m):	12.69	12.69
	Depth at Toe (m):	2.73	2.73
	K1 Shape Coef:	0.82 - Vert. with wing walls	
	Degree of Skew (degrees):	90.00	90.00
	K2 Skew Coef:	1.00	1.00
	Projected Length L' (m):	12.69	12.69
	Avg Depth Obstructed Ya (m):	2.04	2.04
	Flow Obstructed Qe (m3/s):	115.22	115.22
	Area Obstructed Ae (m2):	25.90	25.90
Results			
	Scour Depth Ys (m):	10.34	10.34
	Qe/Ae = Ve:	4.45	4.45
	Froude #:	0.99	0.99

Equation:	Froehlich	Froehlich

Combined Scour Depths

Pier Scour + Contraction Scour (m):		
	Channel:	1.15
Left abutment scour + contraction scour (m):	10.35	
Right abutment scour + contraction scour (m):	10.35	


Bridge Scour RS = 100
Legend
WS PF 1
Ground
Ineff
Bank Sta
Contr Scour
Total Scour
Elevation (m)
1822
1820
1818
1816
1814
1812
1810
1808
1806
-40
-30
-20
-10
0
10
20
30
40
Station (m)

Contraction Scour

		Left	Channel	Right
Input Data				
	Average Depth (m):		1.76	
	Approach Velocity (m/s):		4.12	
	Br Average Depth (m):		2.00	
	BR Opening Flow (m3/s):		185.30	
	BR Top WD (m):		32.12	
	Grain Size D50 (mm):		5.70	
	Approach Flow (m3/s):		185.30	
	Approach Top WD (m):		25.58	
	K1 Coefficient:		0.640	
Results				
	Scour Depth Ys (m):		0.00	
	Critical Velocity (m/s):		1.21	
	Equation:		Live	

Pier Scour

All piers have the same scour depth

Input Data		
	Pier Shape:	Round nose
	Pier Width (m):	1.50
	Grain Size D50 (mm):	5.70000
	Depth Upstream (m):	2.12
	Velocity Upstream (m/s):	2.18
	K1 Nose Shape:	1.00
	Pier Angle:	0.00
	Pier Length (m):	10.95
	K2 Angle Coef:	1.00
	K3 Bed Cond Coef:	1.10
	Grain Size D90 (mm):	100.00000
	K4 Armouring Coef:	0.40
Results		
	Scour Depth Ys (m):	1.08
	Froude #:	0.48
	Equation:	CSU equation

Abutment Scour

		Left	Right
Input Data			
	Station at Toe (m):	-17.51	17.51
	Toe Sta at appr (m):	93.38	94.23
	Abutment Length (m):	12.37	12.37
	Depth at Toe (m):	1.98	1.98
	K1 Shape Coef:	0.82 - Vert. with wing walls	
	Degree of Skew (degrees):	90.00	90.00
	K2 Skew Coef:	1.00	1.00
	Projected Length L' (m):	12.37	12.37
	Avg Depth Obstructed Ya (m):	1.76	1.76
	Flow Obstructed Qe (m3/s):	89.57	89.57
	Area Obstructed Ae (m2):	21.74	21.74
Results			
	Scour Depth Ys (m):	9.30	9.30
	Qe/Ae = Ve:	4.12	4.12
	Froude #:	0.99	0.99

General Scour Calculations					
Bridge 10 Concrete Apron Scour Calculations					
National Engineering Handbook, Part 654, Technical Supplement 14B					
Equation TS14B-23					
	Qd (m3/s)	185.3			
	Wf (m)	31.06			
	d50 (mm)	5.7			
Lacey	K	0.389		Right Angle Bend	
	a	0.333333			
	b	0			
	c	-0.16667			
Blench	K	1.105		Right Angle Bend	
	a	0.666667			
	b	-0.66667			
	c	-0.1092			
General Scour					
Lacey	Z (m)	1.659			
Blench	Z (m)	3.006			

	Bridge 10 Uplift Resistance Calculations										
	Comparison of Uplift Pressure v. Weight of Water+Concrete										
				Unit Weight Water	9.81	kN/m3					
				Unit Weight Concrete	23.6	kN/m3					
				Area	1	m2					
				Concrete Thickness	8	in					
				Concrete Thickness	0.2032	m					
				Weight of Concrete	4.80	kN/m2					
Minimum Desired Factor of Safety for Design Flow and Lower Flows					1.5						
	BRIDGE 100 UPSTREAM										
	Q (m3/2)	WSE	Mat Elev.	Depth of Water (m)	Weight of Water (kN/m2)	Velocity (m/s)	Velocity (ft/s)	Uplift Head (ft)	Uplift Head (m)	Uplift Pressure (kN/m2)	Factor of Safety
50-Yr	185.3	1818.89	1816.31	2.58	25.3	3.27	10.7	2.5	0.8	7.5	4.0
	150	1818.58	1816.31	2.27	22.3	3.01	9.9	2.2	0.7	6.5	4.1
	100	1818.42	1816.31	2.11	20.7	2.56	8.4	1.7	0.5	5.1	5.0
	75	1817.81	1816.31	1.5	14.7	2.27	7.4	1.4	0.4	4.2	4.7
	50	1817.5	1816.31	1.19	11.7	1.92	6.3	1.1	0.3	3.2	5.1
	BRIDGE 100 DOWNSTREAM										
	Q (m3/2)	WSE	Mat Elev.	Depth of Water (m)	Weight of Water (kN/m2)	Velocity (m/s)	Velocity (ft/s)	Uplift Head (ft)	Uplift Head (m)	Uplift Pressure (kN/m2)	Factor of Safety
	185.3	1817.46	1815.51	1.95	19.1	4.34	14.2	4.0	1.2	11.9	2.0
	150	1817.2	1815.51	1.69	16.6	4.05	13.3	3.6	1.1	10.6	2.0
	100	1816.79	1815.51	1.28	12.6	3.55	11.6	2.9	0.9	8.5	2.0
	75	1816.57	1815.51	1.06	10.4	3.23	10.6	2.4	0.7	7.3	2.1
	50	1816.32	1815.51	0.81	7.9	2.82	9.3	2.0	0.6	5.9	2.2
	SECTION 95										
	Q (m3/2)	WSE	Mat Elev.	Depth of Water (m)	Weight of Water (kN/m2)	Velocity (m/s)	Velocity (ft/s)	Uplift Head (ft)	Uplift Head (m)	Uplift Pressure (kN/m2)	Factor of Safety
	185.3	1817.23	1815.5	1.73	17.0	3.97	13.0	3.4	1.0	10.3	2.1
	150	1817.02	1815.5	1.52	14.9	3.71	12.2	3.1	0.9	9.2	2.1
	100	1816.66	1815.5	1.16	11.4	3.28	10.8	2.5	0.8	7.5	2.2
	75	1816.45	1815.5	0.95	9.3	3.03	9.9	2.2	0.7	6.6	2.1
	50	1816.24	1815.5	0.74	7.3	2.62	8.6	1.8	0.5	5.2	2.3
	SECTION 113										
	Q (m3/2)	WSE	Mat Elev.	Depth of Water (m)	Weight of Water (kN/m2)	Velocity (m/s)	Velocity (ft/s)	Uplift Head (ft)	Uplift Head (m)	Uplift Pressure (kN/m2)	Factor of Safety
	185.3	1819.17	1816.3	2.87	28.2	2.3	7.5	1.4	0.4	4.3	7.7
	150	1818.82	1816.3	2.52	24.7	2.15	7.1	1.3	0.4	3.9	7.7
	100	1818.26	1816.3	1.96	19.2	1.88	6.2	1.1	0.3	3.1	7.6
	75	1817.95	1816.3	1.65	16.2	1.7	5.6	0.9	0.3	2.7	7.8
	50	1817.6	1816.3	1.3	12.8	1.46	4.8	0.7	0.2	2.2	8.1

Appendix B

Structural Calculations

GARDEZ TO KHOST
BRIDGE NO. 10

TABLE OF CONTENTS

SUPERSTRUCTURE DEAD LOAD

LENGTH

TOTAL LENGTH = 24.340m
OF SPANS = 2
SPAN LENGTH = 12.170 m = 39.92'

WIDTH

TOTAL WIDTH = 10.950m
ROADWAY WIDTH = 8.0m

DECK SLAB

DECK W = 10.95 m = 35.92'
L = 24.34 m = 79.84' ⇒ V = 6113.21 FT^3
H = 0.650 m = 2.13'

V = 6113.21 FT^3
γ_{CONC} = 150 lb/FT^3
DL_{SLAB} = 916,982.1 lbs = 916.9 KIPS $\boxed{\text{SAY} \Rightarrow DL_{SLAB} = 920 \text{ KIPS}}$

WEARING SURFACE

W.S. W = 8.0 m = 26.24'
L = 24.34 m = 79.84' ⇒ V = 371.04 FT^3
H = 0.054 m = 0.18'

V = 371.04 FT^3
γ_{WS} = 165 lb/FT^3
DL_{WS} = 61,222.3 lbs = 61.2 KIPS $\boxed{\text{SAY} \Rightarrow DL_{WS} = 65 \text{ KIPS}}$

SIDEWALK

SIDEWALK W = 1.2 m = 3.94'
L = 24.34 m = 79.84' ⇒ V = 283.44 FT^3 X 2 = 566.87 FT^3
H = 0.275m = 0.90'

V = 566.87 FT^3
γ_{SW} = 150 lb/FT^3
DL_{SW} = 85031.0 lbs = 85.03 KIPS $\boxed{\text{SAY} \Rightarrow DL_{SW} = 90 \text{ KIPS}}$

TETRA TECH
One Grant Street
Framingham, MA 01701-9005
(508) 903-2000

JOB KHOST #10
SHEET NO. 1 OF 2
CALCULATED BY aen DATE 4/2/14
CHECKED BY SAM DATE 6/25/14
SCALE

SUPERSTRUCTURE DEAD LOAD CONT.

BARRIER / RAIL

BARRIER
W = 0.225 m = 0.74'
L = 24.34 m = 79.84' ⇒ V = 270.55 FT^3 × 2 = 541.11 FT^3
H = 1.4 m = 4.59'

V = 541.11 FT^3
γ_{CONC} = 150 lbs/FT^3
DL_{BAR} = 81166 lbs = 81.17 KIPS

$\boxed{\text{SAY} \Rightarrow DL_{BARRIER} = 85 \text{ KIPS}}$

SUMMARY OF SUPERSTRUCTURE DEAD LOADS

DEAD LOAD	KIPS	SAY	DC	DW
DECK SLAB	917	920 K	920	
WEARING SURFACE	62	65 K		65
SIDEWALK	86	90 K	90	
BARRIER	82	85 K	85	
			1095 K	65 K
TOTAL DL-SUPER		1160 KIPS		

DC 1095 K
DW 65 K

TETRA TECH
One Grant Street
Framingham, MA 01701-9005
(508) 903-2000

JOB KHOST #10
SHEET NO. 2 OF 2
CALCULATED BY alh DATE 4/2/14
CHECKED BY SAM DATE 6/25/14
SCALE

FOR PRELIMINARY DESIGN ONLY

11/6/2013 11:51 AM

SEISMIC DESIGN CATEGORY (SDC)

PGA =	0.29	g	* SEE ATTACHED HAZARD MAPS
S_S, 0.2 SEC =	0.64	g	* SEE ATTACHED HAZARD MAPS
S_1, 1.0 SEC =	0.47	g	* SEE ATTACHED HAZARD MAPS

Table 3.4.2.3-2—Values of F_v as a Function of Site Class and Mapped 1-sec Period Spectral Acceleration Coefficient

Site Class	Mapped Spectral Response Acceleration Coefficient at 1-sec Periods				
	$S_1 < 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \geq 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	*	*	*	*	*

Note: Use straight line interpolation for intermediate values of S_1, where S_1 is the spectral acceleration coefficient at 1.0 sec obtained from the ground motion maps.

* Site-specific response geotechnical investigation and dynamic site response analyses should be considered (Article 3.4.3)

Assume Site Class C or D

Fv (Range for Site Class C & D) =	1.3	–>	1.6		
Fv (Range for Site Class C & D) =	1.33	–>	1.53	<--- Range for S_1 =	0.47

Say Fv = 1.53

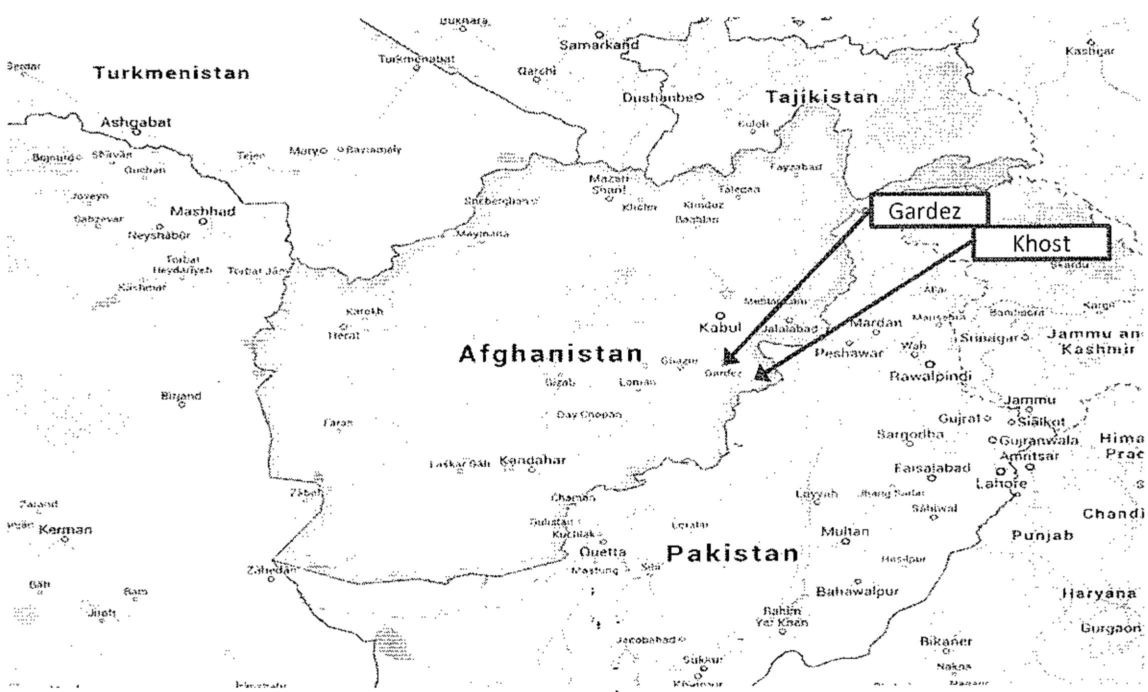
SD1 = 0.72

$$S_{D1} = F_v S_1 \quad (3.4.1\text{-}3)$$

Table 3.5-1—Partitions for Seismic Design Categories A, B, C, and D

Value of $S_{D1} = F_v S_1$	SDC
$S_{D1} < 0.15$	A
$0.15 \leq S_{D1} < 0.30$	B
$0.30 \leq S_{D1} < 0.50$	C
$0.50 \leq S_{D1}$	D

Sesimic Design Cateogry , SDC = D



SEISMIC DESIGN CATEGORY (SDC)

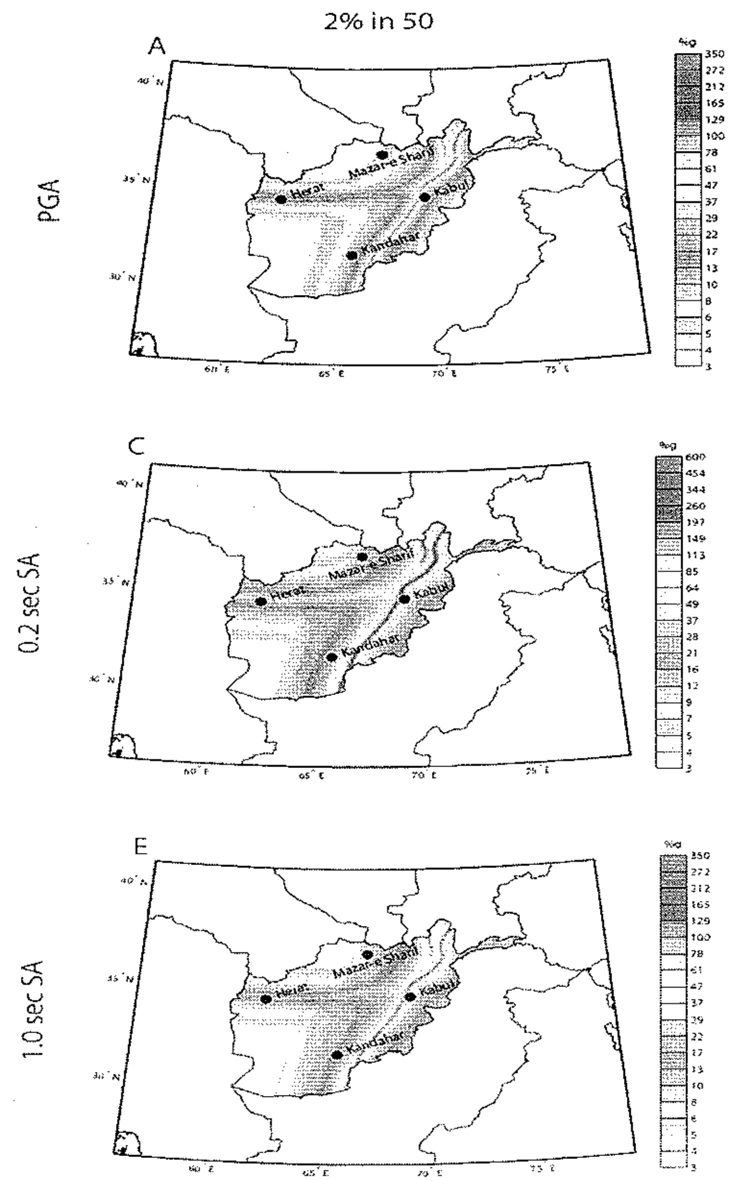
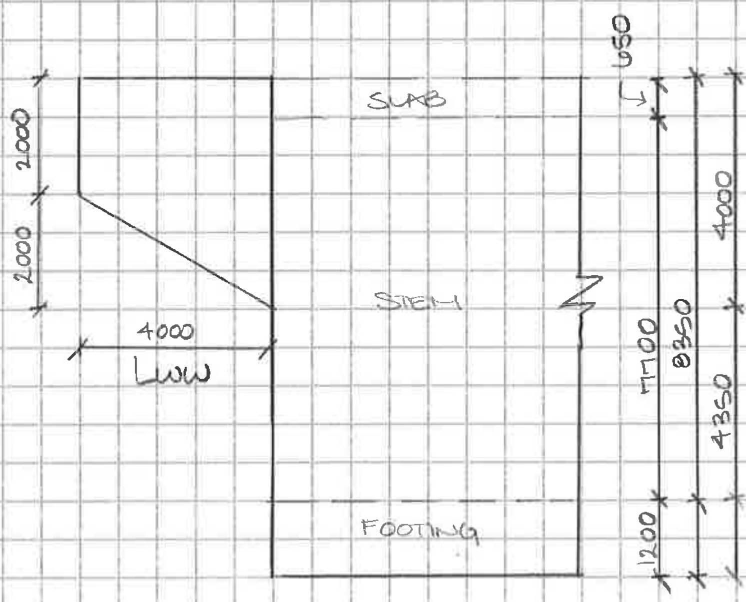
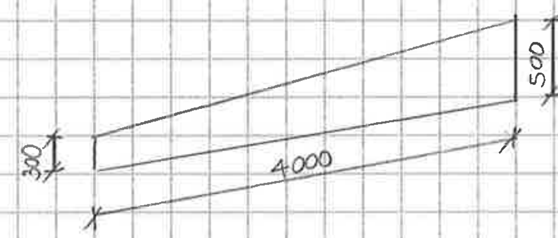


Figure 7. Ground motions for fault sources for PGA (A, B), 0.2-second SA (C, D), and 1.0-second SA (E, F) at 2-percent (A, C, E) and 10-percent (B, D, F) probability of exceedance in 50 years.

EARTH LOAD ON ABUTMENT DUE TO WINGWALL



ELEVATION VIEW
N.T.S.



PLAN VIEW
N.T.S.

HEIGHT OF SOIL = HEIGHT OF WALL

$H_{SOIL} = 4.0m = 13.0'$
$H_{WALL} = 4.0m = 13.0'$
$P_a = \frac{1}{2} \cdot K_E \cdot \gamma \cdot H^2$; $K_E = 0.320$ $\gamma = 130$ PSF

$P_a = 0.5 * 0.320 * 130 * 13^2 = 3515.2$ lbs $= 3.52$ KIPS
P_a (KLF) $= P_a$ (KIPS) $* L_{WW} / L_{ABUT}$ $L_{ABUT} = 10950$ MM $= 35.92'$
$P_a = 3.52$ KIPS $* 13' / 35.92' = 1.27$ KLF

FOR BEARING: $\phi = 31°$ $\phi/3 = 10.33°$
$P_{aH} = P_a \cdot \cos(\phi/3) = 1.27$ KLF $* \cos(10.33) = 1.25$ KLF

	Pa(H) K	ARM FT	Mo K-FT
HORIZONTAL	1.25	22.6	
X 2 WALLS	2.5	22.6	56.5

INPUT FOR EH: WINGWALL ON PRIM LOAD TAB

TETRA TECH
One Grant Street
Framingham, MA 01701-9005
(508) 903-2000

JOB KHDST #10 EARTH LOAD DUE TO WW
SHEET NO. ______ OF ______
CALCULATED BY AWH DATE 4.7.14
CHECKED BY SAM DATE 6/25/14
SCALE N.T.S.

BACKUP CALCULATIONS TO BE USED IN THE INPUT TAB

ABUTMENT LOADING CALCULATIONS - LIVE LOAD (LL), BREAKING FORCE (BR) & SEISMIC LOAD (EQ)

GENERAL INFORMATION

Project Number:	1298\127-1298-12001-LT0077	Designed By:	alh
Description:	Khost Bridge No. 10	Checked By:	SAM
Structure:	Abutment	Date:	6/25/2014

References: AASHTO LRFD Bridge Design Specifications, 6th Edition, 2012

Notes: This spreadsheet computes the loads on an abutment, considering the spans left or right of the abutment is simply supported.

SUPERSTRUCTURE LOADING ON ABUTMENT - VERTICAL FORCES (CONT.)

Live Load, LL

Type of Truck: HL-	93	
Roadway Width =	26.24	ft
Lane Width =	12	ft
Roadway / Lane Width =	2.19	
Use --> No of Lanes =	2	
Multiple Presence Factor, m =	1	

Table 3.6.1.1.2-1—Multiple Presence Factors, *m*

Number of Loaded Lanes	Multiple Presence Factors, *m*
1	1.20
2	1.00
3	0.85
>3	0.65

Truck Loading:

Section 3.6.1.2.2

	Left/Right Span		
Span Length, L =	39.92	ft	
Dynamic Load Allowance, (IM) =	1.33		
Number of Lanes =	2		
Multiple Presence Factor, m =	1.00		
Vmax =	55.2	kips / Lane	<-- T3.3.1.1 Shear & End Reactions
Vmax =	110.40	kips	<- Vmax * m * # of lanes
Reaction, LL V =	110.40	kips	
Reaction, (LL+IM) V =	146.8	kips	<-- IM * V
Total Reaction, Truck (LL) =	110.4	kips	
Total Reaction, Truck (LL+IM) =	146.8	kips	

Section 3.6.2.1

Section 3.6.1.1.2

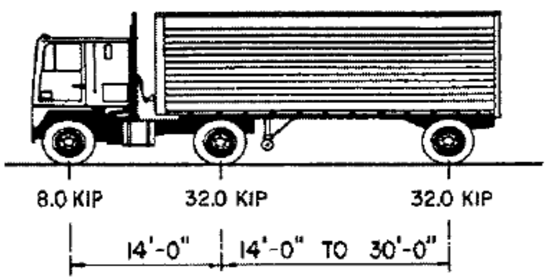


Figure 3.6.1.2.2-1

BACKUP CALCULATIONS TO BE USED IN THE INPUT TAB

ABUTMENT LOADING CALCULATIONS - LIVE LOAD (LL), BREAKING FORCE (BR) & SEISMIC LOAD (EQ)

GENERAL INFORMATION

Project Number:	1298\127-1298-12001-LT0077	Designed By:	alh
Description:	Khost Bridge No. 10	Checked By:	SAM
Structure:	Abutment	Date:	6/25/2014

SUPERSTRUCTURE LOADING ON ABUTMENT - VERTICAL FORCES (CONT.)

Tandem Loading:

Section 3.6.1.2.3

	Left/Right Span			
L =	39.92	ft		
Dynamic Load Allowance, (IM) =	1.33			Section 3.6.2.1
Number of Lanes =	2			
Multiple Presence Factor, m =	1.00			Section 3.6.1.1.2
P1 =	25	kips		
P2 =	25	kips		
Axle Spacing =	4	ft		
Vmax =	47.49	kips/ Lane		
Vmax =	94.99	Kips	<- Vmax * m * # of lanes	
Reaction, LL V =	94.99	kips		
Reaction, (LL+IM) V =	126.34	kips	<-- IM * V	
Total Reaction, Tandem (LL) =	95.0	kips		
Total Reaction, Tandem (LL+IM) =	126.3	kips		

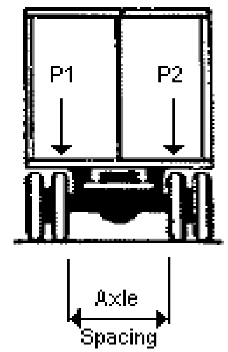


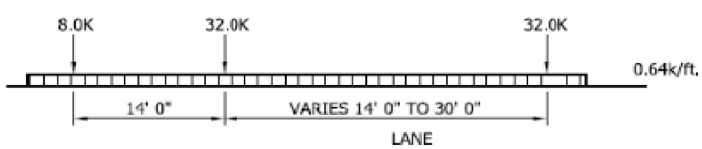
Figure 3.6.1.2.2-1

Live Load, LL (cont.)

Lane Loading:

Section 3.6.1.2.4

	Left/Right Span			
L =	39.92	ft		
Number of Lanes =	2			
Multiple Presence Factor, m =	1.00			Section 3.6.1.1.2
Lane Load =	0.64	klf		
Vmax =	12.77	kips/ Lane		
Vmax =	25.55	Kips	<- Vmax * m * # of lanes	
Reaction, Lane Load (LL) =	25.5	kips		
Total Reaction, Lane Load (LL) =	25.5	kips		



BACKUP CALCULATIONS TO BE USED IN THE INPUT TAB

ABUTMENT LOADING CALCULATIONS - LIVE LOAD (LL), BREAKING FORCE (BR) & SEISMIC LOAD (EQ)

GENERAL INFORMATION

Project Number:	1298\127-1298-12001-LT0077	Designed By:	alh
Description:	Khost Bridge No. 10	Checked By:	SAM
Structure:	Abutment	Date:	6/25/2014

SUPERSTRUCTURE LOADING ON ABUTMENT - VERTICAL FORCES (CONT.)

Pedestrian Live Load

Pedestrian Live Load, PL = 0.075 ksf <--- per AASHTO 3.6.1.6 for Sidewalks with a Width >= 2.0 ft

Width of Sidewalk = 3.94 ft

PL = 0.295 klf

Length of Sidewalk = 39.92 ft

PL = 11.78 kips

--> PL / Abutment = 5.89 kips

Abutment Length = 35.92 ft

--> PL / LF of Abutment = 0.16 klf / Sidewalk

No of Sidewalks = 2

--> PL / LF of Abutment = 0.33 klf

Live Loads

	LL	IM	LL + IM
Truck	55.20	1.33	73.42
Tandem	47.49	1.33	63.17
Lane	12.77	1	12.77
Truck + Lane	67.97		86.19
Tandem + lane	60.27		75.94
Max	67.97		86.19

Max = 86.19 kips

No of Lanes = 2.00

m = 1.00

LL+I = 172.38 kips

Abutment Length = 35.92 ft

LL+ I = 4.80 klf

LL + I + PL = 5.13 klf <-- INPUT Vehicle + Pedestrian Reaction per Linear Foot of Abutment

BACKUP CALCULATIONS TO BE USED IN THE INPUT TAB

ABUTMENT LOADING CALCULATIONS - LIVE LOAD (LL), BREAKING FORCE (BR) & SEISMIC LOAD (EQ)

GENERAL INFORMATION

Project Number:	1298\127-1298-12001-LT0077	Designed By:	alh
Description:	Khost Bridge No. 10	Checked By:	SAM
Structure:	Abutment	Date:	6/25/2014

SUPERSTRUCTURE LOADING ON ABUTMENT - LATERAL FORCES

Braking Force, BR

Section 3.6.4

Notes: Dynamic Load Allowance increase not required. AASHTO3.6.2.1
Braking Force ONLY applies to fixed bearings
Braking Force includes multiple presence factor

Type of Bearing: Fixed

25% Axle Weight of Design Truck =	25%	18.00	kips	Design Truck Axle Weight =	72
25% Axle Weight of Design Tandem =	25%	12.50	kips	Design Tandem Axle Weight =	50
5% (Axle Weight of Design Truck + Lane Load) =	5%	4.88	kips	Design Truck + Lane Axle Weight =	97.55
5% (Axle Weight of Design Tandem Load + Lane Load) =	5%	3.78	kips	Design Tandem + Lane Axle Weight =	75.55

Braking Force on Abutment (BR) =	18	kips	<---- 25% Axle Weight of Design Truck	
Number of Lanes =	2			
Multiple Presence Factor, m =	1			
BR =	1.00	klf		
No of Fixed Ends =	2			
BR =	0.50	klf	<--- Breaking Force Per Linear foot of Abutment	<-- Input Load

Location of Load Application = *0.00* *ft above Bridge Seat*

BACKUP CALCULATIONS TO BE USED IN THE INPUT TAB

ABUTMENT LOADING CALCULATIONS - LIVE LOAD (LL), BREAKING FORCE (BR) & SEISMIC LOAD (EQ)

GENERAL INFORMATION

Project Number:	1298\127-1298-12001-LT0077	Designed By:	alh
Description:	Khost Bridge No. 10	Checked By:	SAM
Structure:	Abutment	Date:	6/25/2014

SUPERSTRUCTURE LOADING ON ABUTMENT/PIER - LATERAL FORCES - EQ

Concrete Pryout (in Tension of a Single Anchor)

$$Nb = Kc * \text{Sqrt}(f'c) * (hef)^{1.6}$$

Kc =	24			
f'c =	3000	psi		
hef =	11.81	in		
Nb =	53351.51	lbs	53.35	kips
Anc =	675000	mm^2		
width =	900	mm		
Length =	750	mm		
Anco =	810000	mm^2		
width =	900	mm		
Length =	900	mm		
Anc / Anco =	0.83			

$$Ncb = (Anc / Anco) * \psi ed,N * \psi c,N * \psi cp,N * Nb$$

ψed,N =	0.90	= 0.7 + 0.3 (Ca,min / 1.5*hef)
Ca,min =	300	mm
1.5 * hef =	450.0	mm
ψc,N =	1.25	
ψcp,N =	1	
ϕ =	0.75	
Ncb =	50.02	kip / Anchor
ϕNcb =	37.51	kip / Anchor

GENERAL INFORMATION

Project Number:	1298\127-1298-12001-LT0077	Designed By:	alh
Description:	Khost Bridge No. 10	Checked By:	SAM
Structure:	Abutment	Date:	6/25/2014

SUPERSTRUCTURE LOADING ON ABUTMENT/PIER - LATERAL FORCES - EQ (CONT.)

Concrete Breakout (in shear of a single anchor)

Vb = 7 (le / do)$^{0.2}$ * sqrt (do) * sqrt (f'c) * (Ca1) $^{1.5}$ Ref: ACI 318 Eq (D-24)

do =	0.985	in	25.0	mm
hef =	11.81	in	300.0	mm
8 *do =	7.88	in	200.2	mm
le =	7.88	in	200.2	mm
Ca1 =	13.78	in	350.0	mm
Vb =	29503.16	lbs	29.50	kips

Vcb = (Anc / Anco) * ψed,V * ψc,V * Vb

Anc / Anco =	1
ψed,V =	1
ψc,V=	1
∅ =	0.75

Vcb =	29.50	kip / Anchor
∅ Vcb =	22.13	kip / Anchor

	kips / Anchor	No of Anchors	Kips	
Concrete Pryout	37.51	0 in Tension	0.00	<-- 0 Anchors in Tension for Abutment
Concrete Breakout	22.13	10 in Shear	221.27	
Total EQ on Superstructure			221.27	Kips

GENERAL INFORMATION

Project Number:	1298\127-1298-12001-LT0077	Designed By:	alh
Description:	Khost Bridge No. 10	Checked By:	SAM
Structure:	Abutment	Date:	6/25/2014

AASHTO Standard Specifications for Highway Bridges - 17th edition 2002

Loading -- HS 20-44 (MS18)

TABLE OF MAXIMUM MOMENTS, SHEARS, AND REACTIONS— SIMPLE SPANS, ONE LANE

Spans in feet; moments in thousands of foot-pounds; shears and reactions in thousands of pounds.

These values are subject to specification reduction for loading of multiple lanes.

Impact not included.

Span	Moment	End shear and end reaction (a)	Span	Moment	End shear and end reaction (a)
1	8.0(b)	32.0(b)	42	485.3(b)	56.0(b)
2	16.0(b)	32.0(b)	44	520.9(b)	56.7(b)
3	24.0(b)	32.0(b)	46	556.5(b)	57.3(b)
4	32.0(b)	32.0(b)	48	592.1(b)	58.0(b)
5	40.0(b)	32.0(b)	50	627.9(b)	58.5(b)
6	48.0(b)	32.0(b)	52	663.6(b)	59.1(b)
7	56.0(b)	32.0(b)	54	699.3(b)	59.6(b)
8	64.0(b)	32.0(b)	56	735.1(b)	60.0(b)
9	72.0(b)	32.0(b)	58	770.8(b)	60.4(b)
10	80.0(b)	32.0(b)	60	806.5(b)	60.8(b)
11	88.0(b)	32.0(b)	62	842.4(b)	61.2(b)
12	96.0(b)	32.0(b)	64	878.1(b)	61.5(b)
13	104.0(b)	32.0(b)	66	914.0(b)	61.9(b)
14	112.0(b)	32.0(b)	68	949.7(b)	62.1(b)
15	120.0(b)	34.1(b)	70	985.6(b)	62.4(b)
16	128.0(b)	36.0(b)	75	1,075.1(b)	63.1(b)
17	136.0(b)	37.7(b)	80	1,164.9(b)	63.6(b)
18	144.0(b)	39.1(b)	85	1,254.7(b)	64.1(b)
19	152.0(b)	40.4(b)	90	1,344.4(b)	64.5(b)
20	160.0(b)	41.6(b)	95	1,434.1(b)	64.9(b)
21	168.0(b)	42.7(b)	100	1,524.0(b)	65.3(b)
22	176.0(b)	43.6(b)	110	1,703.6(b)	65.9(b)
23	184.0(b)	44.5(b)	120	1,883.3(b)	66.4(b)
24	192.7(b)	45.3(b)	130	2,063.1(b)	67.6
25	207.4(b)	46.1(b)	140	2,242.8(b)	70.8
26	222.2(b)	46.8(b)	150	2,475.1	74.0
27	237.0(b)	47.4(b)	160	2,768.0	77.2
28	252.0(b)	48.0(b)	170	3,077.1	80.4
29	267.0(b)	48.8(b)	180	3,402.1	83.6
30	282.1(b)	49.6(b)	190	3,743.1	86.8
31	297.3(b)	50.3(b)	200	4,100.0	90.0
32	312.5(b)	51.0(b)	220	4,862.0	96.4
33	327.8(b)	51.6(b)	240	5,688.0	102.8
34	343.5(b)	52.2(b)	260	6,578.0	109.2
35	361.2(b)	52.8(b)	280	7,532.0	115.6
36	378.9(b)	53.3(b)	300	8,550.0	122.0
37	396.6(b)	53.8(b)			
38	414.3(b)	54.3(b)			
39	432.1(b)	54.8(b)			
40	449.8(b)	55.2(b)			

GENERAL INFORMATION

Project Number:	1298\127-1298-12001-LT0077	Designed By:	alh
Description:	Khost Bridge No. 10	Checked By:	SAM
Structure:	Abutment	Date:	6/25/2014

Table 3.3.1.1
Maximum Unfactored HL-93 Live Load Moments, Shears, and Reactions
Simple Spans, One Lane, w/o Dynamic Load Allowance

	MOMENTS					SHEARS & END REACTIONS			
SPAN FT	TRUCK KIP-FT	TANDEM KIP-FT	LANE KIP-FT	TOTAL KIP-FT	SPAN PT. %	TRUCK KIP	TANDEM KIP	LANE KIP	TOTAL KIP
1	8.0	6.3	0.1	8.1	0.50	32.0	25.0	0.3	32.3
2	16.0	12.5	0.3	16.3	0.50	32.0	25.0	0.6	32.6
3	24.0	18.8	0.7	24.7	0.50	32.0	25.0	1.0	33.0
4	32.0	25.0	1.3	33.3	0.50	32.0	25.0	1.3	33.3
5	40.0	31.3	2.0	42.0	0.50	32.0	30.0	1.6	33.6
6	48.0	37.5	2.9	50.9	0.50	32.0	33.3	1.9	35.3
7	56.0	43.8	3.9	59.9	0.50	32.0	35.7	2.2	38.0
8	64.0	50.0	5.1	69.1	0.50	32.0	37.5	2.6	40.1
9	72.0	62.5	6.5	78.5	0.50	32.0	38.9	2.9	41.8
10	80.0	75.0	8.0	88.0	0.50	32.0	40.0	3.2	43.2
11	84.5	92.0	9.3	101.3	0.40	32.0	40.9	3.5	44.4
12	92.2	104.0	11.1	115.1	0.40	32.0	41.7	3.8	45.5
13	103.0	115.9	13.4	129.3	0.45	32.0	42.3	4.2	46.5
14	110.9	128.3	15.5	143.8	0.45	32.0	42.9	4.5	47.3
15	118.8	140.6	17.8	158.4	0.45	34.1	43.3	4.8	48.1
16	126.7	153.0	20.3	173.3	0.45	36.0	43.8	5.1	48.9
17	134.6	165.4	22.9	188.3	0.45	37.6	44.1	5.4	49.6
18	142.6	177.8	25.7	203.4	0.45	39.1	44.4	5.8	50.2
19	150.5	190.1	28.6	218.7	0.45	40.4	44.7	6.1	50.8
20	158.4	202.5	31.7	234.2	0.45	41.6	45.0	6.4	51.4
21	166.3	214.9	34.9	249.8	0.45	42.7	45.2	6.7	52.0
22	174.2	227.3	38.3	265.6	0.45	43.6	45.5	7.0	52.5
23	182.2	239.6	41.9	281.5	0.45	44.5	45.7	7.4	53.0
24	190.1	252.0	45.6	297.6	0.45	45.3	45.8	7.7	53.5
25	198.0	264.4	49.5	313.9	0.45	46.1	46.0	8.0	54.1
26	210.2	276.8	53.5	330.3	0.45	46.8	46.2	8.3	55.1
27	226.1	289.1	57.7	346.9	0.45	47.4	46.3	8.6	56.0
28	241.9	301.5	62.1	363.6	0.45	48.0	46.4	9.0	57.0
29	257.8	313.9	66.6	380.5	0.45	48.8	46.6	9.3	58.1
30	273.6	326.3	71.3	397.5	0.45	49.6	46.7	9.6	59.2
31	289.4	338.6	76.1	414.7	0.45	50.3	46.8	9.9	60.2
32	307.0	351.0	81.1	432.1	0.45	51.0	46.9	10.2	61.2
33	324.9	363.4	86.2	449.6	0.45	51.6	47.0	10.6	62.2
34	332.0	375.0	92.5	467.5	0.50	52.2	47.1	10.9	63.1
35	350.0	387.5	98.0	485.5	0.50	52.8	47.1	11.2	64.0
36	368.0	400.0	103.7	503.7	0.50	53.3	47.2	11.5	64.9
37	386.0	412.5	109.5	522.0	0.50	53.8	47.3	11.8	65.7
38	404.0	425.0	115.5	540.5	0.50	54.3	47.4	12.2	66.5
39	422.0	437.5	121.7	559.2	0.50	54.8	47.4	12.5	67.2
40	440.0	450.0	128.0	578.0	0.50	55.2	47.5	12.8	68.0

GENERAL INFORMATION

Project Number:	1298\127-1298-12001-LT0077	Designed By:	alh
Description:	Khost Bridge No. 10	Checked By:	SAM
Structure:	Abutment	Date:	6/25/2014

LRFD BRIDGE DESIGN 3-9

Table 3.3.1.2

Maximum Unfactored HL-93 Live Load Moments, Shears, and Reactions
Simple Spans, One Lane, w/o Dynamic Load Allowance

	MOMENTS					SHEARS & END REACTIONS			
SPAN FT	TRUCK KIP-FT	TANDEM KIP-FT	LANE KIP-FT	TOTAL KIP-FT	SPAN PT. %	TRUCK KIP	TANDEM KIP	LANE KIP	TOTAL KIP
42	485.2	474.8	139.7	624.9	0.45	56.0	47.6	13.4	69.4
44	520.9	499.5	153.3	674.2	0.45	56.7	47.7	14.1	70.8
46	556.5	524.3	167.6	724.1	0.45	57.4	47.8	14.7	72.1
48	592.2	549.0	182.5	774.6	0.45	58.0	47.9	15.4	73.4
50	627.8	573.8	198.0	825.8	0.45	58.6	48.0	16.0	74.6
52	663.4	598.5	214.2	877.6	0.45	59.1	48.1	16.6	75.7
54	699.1	623.3	230.9	930.0	0.45	59.6	48.1	17.3	76.8
56	734.7	648.0	248.4	983.1	0.45	60.0	48.2	17.9	77.9
58	770.4	672.8	266.4	1036.8	0.45	60.4	48.3	18.6	79.0
60	806.0	697.5	285.1	1091.1	0.45	60.8	48.3	19.2	80.0
62	841.6	722.3	304.4	1146.1	0.45	61.2	48.4	19.8	81.0
64	877.3	747.0	324.4	1201.7	0.45	61.5	48.4	20.5	82.0
66	912.9	771.8	345.0	1257.9	0.45	61.8	48.5	21.1	82.9
68	948.6	796.5	366.2	1314.8	0.45	62.1	48.5	21.8	83.9
70	984.2	821.3	388.1	1372.3	0.45	62.4	48.6	22.4	84.8
75	1070.0	887.5	450.0	1520.0	0.50	63.0	48.7	24.0	87.0
80	1160.0	950.0	512.0	1672.0	0.50	63.6	48.8	25.6	89.2
85	1250.0	1012.5	578.0	1828.0	0.50	64.1	48.8	27.2	91.3
90	1340.0	1075.0	648.0	1988.0	0.50	64.5	48.9	28.8	93.3
95	1430.0	1137.5	722.0	2152.0	0.50	64.9	48.9	30.4	95.3
100	1520.0	1200.0	800.0	2320.0	0.50	65.3	49.0	32.0	97.3
110	1700.0	1325.0	968.0	2668.0	0.50	65.9	49.1	35.2	101.1
120	1880.0	1450.0	1152.0	3032.0	0.50	66.4	49.2	38.4	104.8
130	2060.0	1575.0	1352.0	3412.0	0.50	66.8	49.2	41.6	108.4
140	2240.0	1700.0	1568.0	3808.0	0.50	67.2	49.3	44.8	112.0
150	2420.0	1825.0	1800.0	4220.0	0.50	67.5	49.3	48.0	115.5
160	2600.0	1950.0	2048.0	4648.0	0.50	67.8	49.4	51.2	119.0
170	2780.0	2075.0	2312.0	5092.0	0.50	68.0	49.4	54.4	122.4
180	2960.0	2200.0	2592.0	5552.0	0.50	68.3	49.4	57.6	125.9
190	3140.0	2325.0	2888.0	6028.0	0.50	68.5	49.5	60.8	129.3
200	3320.0	2450.0	3200.0	6520.0	0.50	68.6	49.5	64.0	132.6

http://www.dot.nd.gov/manuals/bridge/lrfd-bridge-design/Section03A.pdf

CANTILEVER ABUTMENT DESIGN -INPUT

General Information

Project Number:	1298\127-1298-12001-LT0077	Designed By:	ALH
Description:	Khost Bridge No. 10	Checked By:	SAM
Structure:	Abutment	Date:	June 25, 2014

References:

AASHTO LRFD Bridge Design Specifications, 6th Edition, 2012	AASHTO Guide Specifications for LRFD Seismic Bridge Design 2011
ACI 318-08 Building Code Requirements for Structural Concrete, 2005	
2009 MassDOT LRFD Bridge Manual, including draft Novemeber 2012 provisions	

General Notes: This template assumes that the soils strata behind the abutment is uniform (only 1 strata is considered).

BRIDGE Design | Khost Bridge Notes

Project Notes:

General Design Parameters

Input Section : 1.0

GEOMETRY INFORMATION INPUT:

PROPOSED TOP OF ROADWAY ELEV:				ft		m
PROPOSED TOP OF BACKWALL ELEV:			5971.44	ft	1820.561	m
PROPOSED BRIDGE SEAT ELEV:	H_Backwall = 2.16 ft		5969.28	ft	1819.901	m
PROPOSED TOP OF FOOTING ELEV:	H_Footing = 3.94 ft		5944.02	ft	1812.200	m
PROPOSED BOT. OF FOOTING ELEV:			5940.08	ft	1811.000	m
ELEVATION OF HIGH WATER:		FOR NO WATER = 0.00	5963.70	ft	1818.200	m
PROPOSED BRIDGE SEAT WIDTH:			2.30	ft	0.700	m
PROPOSED BACKWALL WIDTH:			1.64	ft	0.500	m
ABUTMENT/WALL DESIGN LENGTH:	1.00	Actual Length:	35.92	ft	10.950	m
FOOTING LENGTH		Actual Length:	35.92	ft	10.950	m

DW CALCULATION INPUT:

WEARING SURFACE DEPTH:	1.97 IN	x 1. Layers	0.16	ft <-- ---	0.050	m
ROADWAY WIDTH:			26.24	ft	8.000	m
BRIDGE SPAN:	Total Length = 79.84		39.92	ft	12.170	m
NUMBER OF GIRDERS:			1			

MATERIAL PROPERTIES:

CUBIC WEIGHT CONCRETE:	150.00	pcf
COMP. STRENGTH OF CONC. = F'c:	4.00	ksi
MAXIMUM SIZE OF COARSE AGGREGATE	1.50	in
TENSILE STRENGTH OF REBAR = Fy:	60.00	ksi
CUBIC WEIGHT OF HOT MIX ASPHALT (HMA):	165.00	pcf

GEOTECHNICAL INFORMATION:

BEARING RESISTANCE (CAPACITY):	8.00	ksf	<-- Per Geotech Report
NOMINAL BEARING RESISTANCE, qn :	17.78	ksf	<-- Assumed
WEIGHT OF SOIL BACKFILL:	130.00	Lbs/CF	<-- Assumed
WALL ON ROCK?	N	(Y OR N)	
WALL ON PILES?	N	(Y OR N)	
GRAVITY WALL?	N	(Y OR N)	
BETA: SLOPE OF BACKFILL:	0.00	DEG	<-- Assumed
THETA: BATTER ANGLE BACKWALL:	90.00	DEG	AASHTO Table 3.11.5.3-1
PHI: FRICTION ANGLE OF BACKFILL:	33.00	DEG	<-- Assumed
DELTA: ANGLE BACKWALL FRICTION:	22.00	DEG	<-- Assumed δ=2/3 (ϕ)

Fill-in for Abutment / Pier Design

-

CANTILEVER ABUTMENT DESIGN	Y
GRAVITY ABUTMENT DESIGN	N
CANTILEVER WALL DESIGN	N
GRAVITY WALL DESIGN	N
PIER DESIGN	N

CANTILEVER ABUTMENT DESIGN -INPUT

General Information

Project Number:	1298\127-1298-12001-LT0077	Designed By:	ALH
Description:	Khost Bridge No. 10	Checked By:	SAM
Structure:	Abutment	Date:	June 25, 2014

General Loading Parameters

Input Section : 2.0

LIVE LOAD INFORMATION:

APPROACH SLAB:	Y	(Y OR N)
ROADWAY WITHIN H/2 OF TOP OF WALL:	Y	(Y OR N)
Live Load Surcharge to be Considered?:	Y	
SURCHARGE HEIGHT:	2.00	ft REF: Table 3.11.6.4-1
Construction Surcharge, q:	250.00	psf REF: C3.4.2.1

SEISMIC LOAD INFORMATION:

WALL RESTRAINED HORZ. MOVMT.(Y/N):	N	(Y OR N)
SEISMIC ACCELERATION COEFF. A:	0.290	REF: FIG.3.10.2.1-2, AASHTO
SEISMIC CATEGORY:	D	<---Assumed based on Location & AASHTO Seimic Design Guide

RAILING CLASS: S3-TL4 (CT) (PER MASSDOT LRFD BRIDGE MANUAL PART 1) 3.3.2.2 **<--- N/A**

Horizontal Railing Design Load	0.00	kips
Horizontal Railing Impact Length	0.00	ft
Wall Height+Rail Height	0.00	ft
Distributed Horizontal Railing Design Load @ top of wall	0.00	klf
Distributed Horizontal Railing Design Load @ bottom of wall	0.00	klf/wall height
Railing Dead Load	0.00	
Additional Moment From Railing Impact	0.00	<--- Note: The added moment from top of railing to bottom of railing is distributed along bottom of footing*

STREAM PRESSURE

Pmax	0.00	psf
Consider Stream Flow:	N	<--- Do not include stream pressure for the wall.

SURCHARGE HEIGHT (Per ASSHTO 3.11.6.4 Live Load Surchage)

ABUTMENTS (N/A for PIERS) ----> Table 3.11.6.4-1

Table 3.11.6.4-1 - Equivalent Height of Soil for Vehicular Loading on Abutments Perpindicular to Traffic

Abutment Height (ft)	h_{eq} (ft)
5	4
10	3
>20	2

Surcharge Height = 2.00 ft

RETAINING WALLS -- --> Table 3.11.6.4-2

See Table 3.11.6.4-2 for Equivalent Height of Soil for Vechicular Loading on Retaining Walls Parallel to Traffic.

Retaining Wall Height (ft)	heq (ft) Distance from wall backface to edge of traffic.	
	0.0 ft	≥ 1.0 ft
5	5	2
10	3.5	2
>20	2	2

Distance from wall backface to edge of traffic = 0.0 ft

Surcharge Height = 2.00 ft

Note: See 3.11.6.5 for Possible Reduction of Surcharge

CANTILEVER ABUTMENT DESIGN -INPUT

General Information

Project Number:	1298\127-1298-12001-LT0077	Designed By:	ALH
Description:	Khost Bridge No. 10	Checked By:	SAM
Structure:	Abutment	Date:	June 25, 2014

Superstructure Loading Parameters

Input Section : 3.0

ADDITIONAL LOADS ON STRUCTURE

(load is per linear foot of structure (Abutment/ Pier/ Wall) NOT the Footing, arm from front edge of bridge seat)

LOADS		LOAD (klf)	ARM (feet)			
(DC+DW), SUPERSTRUCT. DEAD LOAD:	DL	8.07	1.15	Distance from front face of the abutment/Pier/Wall to CL of bearing	Include =	Y
DC (Structural Components & nonstructural attachments)	DC	7.62	1.15	Distance from front face of the abutment/Pier/Wall to CL of bearing	Include =	Y
DW (Wearing Surface & Utilities)	DW	0.45	1.15	Distance from front face of the abutment/Pier/Wall to CL of bearing	Include =	Y
(LL+IM+PL), LIVE LOAD, IMPACT AND PED LL:	LL+IM+PL	5.13	1.15	Distance from front face of the abutment/Pier/Wall to CL of bearing	Include =	Y
WS, WIND LOAD ON STRUCTURE:	WS	0.00	0.00	Distance above the bridge seat where the longitudinal force is applied.	Include =	Y
WL, WIND LOAD ON LIVE LOAD:	WL	0.00	0.00	Distance above the bridge seat where the longitudinal force is applied.	Include =	Y
BR, BREAKING LOAD :	BR	0.50	0.00	Distance above the bridge seat where the longitudinal force is applied.	Include =	Y
TU, THERMAL FORCE:	TU	0.00	0.00	Distance above the bridge seat where the longitudinal force is applied.	Include =	Y
EQ, SEISMIC LOAD ON SUPERSTRUCTURE:	EQ	6.16	0.00	Distance above the bridge seat where the longitudinal force is applied.	Include =	Y
CT, VEHICLE COLLISION LOAD	CT	0.00	0.00	Distance above top pf wall equal to the height of rail	Include =	Y

Note: Per AASHTO 11.5.1, abutments and retaining walls should be designed for EH, WA, LS, DS, DC, TU, EQ. Therefore, including wind and breaking forces is conservative. Say OK

CANTILEVER ABUTMENT DESIGN -INPUT

General Information

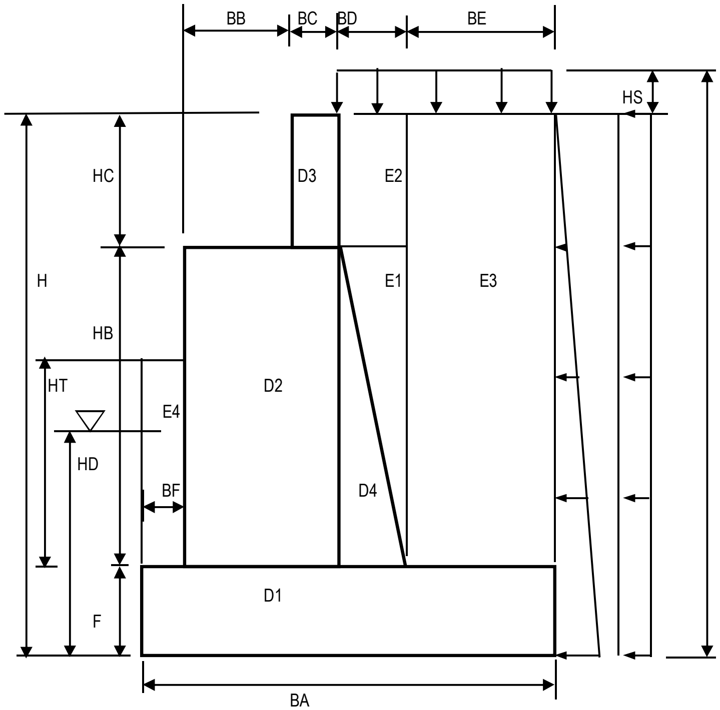
Project Number:	1298\127-1298-12001-LT0077	Designed By:	ALH
Description:	Khost Bridge No. 10	Checked By:	SAM
Structure:	Abutment	Date:	June 25, 2014

Abutment Geometry

Input Section : 4.0

CALCULATION OF WALL AND BACKFILL GEOMETRY:

		Prelim Size	User Adjust	Final Size (ft)	Approx Size (mm)
HEIGHT OF ABUTMENT / WALL, H:	H =	31.360	0.00	31.36	9500
HEIGHT OF FOOTING, F:	F =	3.936	0.00	3.94	1200
HEIGHT OF STEM, HB:	HB =	25.260	0.00	25.26	7600
HEIGHT OF BACKWALL, HC:	HC =	2.165	0.00	2.16	700
HEIGHT OF HIGH WATER, HD:	HD =	23.616	0.00	23.62	7100
HEIGHT OF SURCHARGE, HS:	HS =	2.000	0.00	2.00	600
WIDTH OF FOOTING, BA:	BA =	19.680	0.00	19.68	5910
WIDTH OF BRIDGE SEAT, BB:	BB =	2.296	0.00	2.30	690
WIDTH OF BACKWALL, BC:	BC =	1.640	0.00	1.64	500
WIDTH OF BATTER OF STEM, BD:	BD =	0.000	0.00	0.00	0
WIDTH OF FOOTING HEEL, BE:	BE =	9.180	0.00	9.18	2760
WIDTH OF FOOTING TOE, BF:	BF =	6.564	0.00	6.56	1970
HEIGHT OF SOIL OVER TOE, HT:	HT =	10.860	0.00	10.86	3260
HEIGHT OF SOIL OVER HEEL, HH:	HH =	27.425	0.00	27.42	8300
HEIGHT OF SOIL AT FRONT FACE FACE (TOE), HS1	Hss1 =	14.80		14.80	4500
HEIGHT OF SOIL AT BACKFACE FACE (HEEL), HS2	Hss2 =	31.36		31.36	9500



OVERALL QUANTITIES:

WEIGHT OF CONCRETE WALL/L.F.:	27.065	Kips per l.f.
CONCRETE QUANTITY / L.F.:	6.683	C.Y. per l.f.

SUMMARY OF QUANTITIES:

STEEL / L.F. =	1040.032	LBS/L.F.
CONC. / L.F =	6.683	C.Y./L.F.

Geometry Check: Check Width: ok

Check Height: ok

CANTILEVER ABUTMENT DESIGN - PRIMARY LOADS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	ALH
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Abutment	**Date:**	June 25, 2014

References:
- AASHTO LRFD Bridge Design Specifications, 6th Edition, 2012
- ACI 318-08 Building Code Requirements for Structural Concrete, 2005
- 2009 MassDOT LRFD Bridge Manual, including draft Novemeber 2012 provisions
- AASHTO Guide Specifications for LRFD Seismic Bridge Design 2011

Notes:
This template assumes that the soils strata behind the abutment is uniform (only 1 strata is considered).
BRIDGE Design **Khost Bridge Notes**

Calculate Dead Loads

Primary Loads Section : 1.0

Superstructure Loads: Vertical: Horizontal:

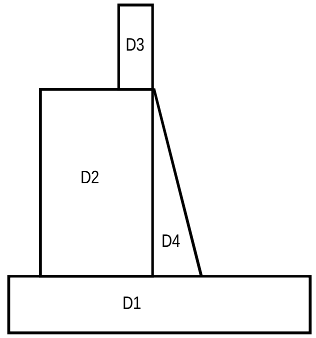
AREA #		Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	Superstructure	7.62	7.71	58.78			
DW	Superstructure	0.45	7.71	3.49			

* See the load column under "Additional Loads on Structure" in the "General Loading Parameters" section for the above forces.

Substructure Loads: Vertical: Horizontal:

AREA #		Volume (CF)	γ_{conc} (pcf)	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	D1	77.46	150.00	11.62	9.84	114.33			
	D2	99.42	150.00	14.91	8.53	127.24			
	D3	3.55	150.00	0.53	9.68	5.15			
	D4	0.00	150.00	0.00	10.50	0.00			
	Subtotal Concrete			**27.07**		**246.73**			

<-- N/A, NO BATTER FOR THIS DESIGN



Total Dead Load: Vertical: Horizontal:

AREA #	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
TOTAL DC (Super + Sub)	**34.69**		**305.51**			
TOTAL DW (Super)	**0.45**		**3.49**			
TOTAL DC (Substr. Only - Construction)	**27.07**		**246.73**			

CANTILEVER ABUTMENT DESIGN - PRIMARY LOADS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	ALH
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Abutment	**Date:**	June 25, 2014

Calculate Earth Loads

Primary Loads Section : 2.0

Compute Horizontal Earth Pressure, EH:

Coulomb's Active Earth Pressure: (per MHD 3.1.5 and AASHTO 3.11.5.3)

PHI, $\phi f'$ =	33.00	Degrees,	Rad =	0.58
DELTA, δ =	22.00	Degrees,	Rad =	0.38
BETA, β =	0.00	Degrees,	Rad =	0.00
THETA, θ =	90.00	Degrees,	Rad =	1.57
Γ (per AASHTO Eq. 3.11.5.3-2)=	2.87			
Ka (per AASHTO Eq. 3.11.5.3-1)=	**0.264**			

Earth Pressure Coefficient to be Used for Design per MassDOT

All Walls on Rock	ko	0.455	
All Walls on Piles	ko	0.455	
Cantilever Walls < than 16' in Height	0.5*(Ko + Ka)	0.360	
Cantilever Walls > than 16' in Height	Ka	0.264	<-- USE
Gravity wall supported on Spread Footing	Ka	0.264	

At-Rest Earth Pressure Coeff:

Ko = **0.455**

Earth Pressure Coefficient to be Used for Design: 'Active pressure coefficients shall be estimated using Coulomb Theory.

WALL ON LEDGE:	N	(Y OR N)
WALL ON PILES:	N	(Y OR N)
Wall Height:	31.36	ft
Earth pressure Type:	Ka	
Ke =	**0.264**	<=== **Does not govern.**

Earth Pressure Coefficients to be Used for Design per Geotechnical Report:

Ko =	0.49	
Ka =	0.32	
Ke (geotech) =	**0.320**	<===== **Governs.**

Compute Lateral Earth Pressure:

Application of lateral earth pressure shall be per AASHTO Figure C3.11.5.3-1. This shows a different application for Gravity and Cantilever (semi-gravity) walls.
Note that the reduction in lateral earth pressures due to the water table is not included in this section. It is included in the WA (Bouyancy) section of this design.

Cantilever (semi-gravity) Walls:

Load inclination from horizontal, min = $\phi/3$ =	11.00	degrees
Load inclination from horizontal, max = $\phi*2/3$ =	22.00	degrees
GAMMA =	130.00	pcf
H = Soil Height at Back face, Hss1	**14.80**	Feet
Lateral Earth Load, Pa = 1/2*Ke*γ*H^2 =	4.55	kips
Arm for Horiz Load above BOF = H/3 =	4.93	ft
Arm for Vert Load from Toe = F =	19.68	ft

Consider minimum inclination for Sliding, Overturning and Bearing Pressure:

Vertical Component, Pav = Pa*sin(ϕ/3) =	0.87	klf
Horizontal Component, Pah = Pa*cos(ϕ/3) =	4.47	klf

Consider maximum inclination for Footing Heel Reinforcement:

Vertical Component, Pav = Pa*sin(ϕ*2/3) =	1.71	klf
Horizontal Component, Pah = Pa*cos(ϕ*2/3) =	4.22	klf

THIS SECTION IS FOR CANTILEVER OR SEMI-GRAVITY WALLS ONLY

Gravity Walls:

Load inclination from horizontal = $\delta + (90-\theta)$ =	22.00	degrees
GAMMA =	130.00	pcf
H =	14.80	Feet
Lateral Earth Load, Pa = 1/2*Ke*γ*H^2 =	4.55	kips
Arm for Horiz Load above BOF = H/3 =	4.93	ft
Arm for Vert Load from Toe=(BF+BB+BC+BD*2/3) =	10.50	ft

Consider for Sliding, Overturning, Bearing Pressure and Footing Reinforcement:

Vertical Component, Pav = Pa*sin($\delta+(90-\theta)$) =	1.71	klf
Horizontal Component, Pah = Pa*cos($\delta+(90-\theta)$) =	4.22	klf
Is the wall a Gravity Wall?	N	

N/A --> THIS SECTION IS FOR GRAVITY WALLS ONLY

CANTILEVER ABUTMENT DESIGN - PRIMARY LOADS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	ALH
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Abutment	**Date:**	June 25, 2014

Calculate Earth Loads Continued..

Primary Loads Section : 2.1

Include Passive Earth Pressure	Y
Pp Factor	1

ϕ = Soil Friction Angle	33.00	degrees
δ = Wall Interface Friction	22.00	degrees = 2/3 * ϕ --> 11.6.5.5
Kp = Passive Earth Pressure Coefficient	3.13	Fig A11.4-2
γ = Unit Weight of Soil	130.00	pcf
H = Hss2= Height of Soil at Front Face - 1'	30.36	ft

Equation A11.4-4 ----> 1/2*γ*Kp*H^2= 187.54 klf > Pah -------> Use Pp = Pah ---------> **P_p = 4.47 klf**

Arm for Horiz Load above BOF = H/3 = **10.12** ft (AASHTO pg 11-112)

CANTILEVER ABUTMENT DESIGN - PRIMARY LOADS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	ALH
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Abutment	**Date:**	June 25, 2014

Calculate Earth Loads Continued..

Primary Loads Section : 2.2

Forces From Earth Retention, EH:

Active Pressure Force Inclination	AREA #	Vertical: Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horizontal: Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
Condition of Minimum Inclination of Active Earth Pressure. (This condition to be used for designs other than heel reinf.)	Forces From Active Earth Pressure	0.87	19.68	17.10	4.47	4.93	22.05
	Forces From Passive Earth Pressure				4.47	10.12	45.24
	EH: Due to Cantilevered Wingwalls (QTY 2)	0.00	0.00	0.00	2.50	22.60	56.50
	When Passive Pressure Considered.	0.87	19.68	17.10	2.50	13.32	33.31
	When Passive Pressure Not Considered.	0.87	19.68	17.10	6.97	11.27	78.55
	Consider Passive Pressure To Counteract The Active Pressure From The Retained Earth						
	Controlling Earth Pressures	**0.87**	**19.68**	**17.10**	**2.50**	**13.32**	**33.31**
Condition of Max Inclination of Active Earth Pressure. (This condition used for heel reinf. Design only)	**Earth Pressures For Heel Reinforcement Design**	**1.71**	**19.68**	**33.57**	**4.22**	**4.93**	**20.82**

<== See attached Calculations: Earth Load on Abutment due to wingwalls

<=== Note, Based on AASHTO Figure C11.5.6-1, both the vertical and horizontal comp

Vertical Earth Pressure, EV:

AREA #		Volume (CF)	γ_{SOIL} (plf)	Vertical: Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horizontal: Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
EV	E1	0.00	130.00	0.00	10.50	0.00			
	E2	0.00	130.00	0.00	10.50	0.00			
	E3	251.75	130.00	32.73	15.09	493.86			
	E4	71.29	130.00	9.27	3.28	30.41			
TOTAL EV				**41.99**		**524.28**			

<-- N/A Batter = 0

<-- N/A Batter = 0

Note, **per AASHTO 11.6.1.2**, the weight of the soil over the battered portion of the stem or over the base of a footing may be considered as part of the effective weight of the abutment. This is consistant with design.

Earth Surcharge, ES: (This applies for construction case only)

q =	250.00	psf
Uniform Load on Wall, p=Ke*q =	0.080	ksf
Wall Height, H =	31.36	Feet
Heel Length, BE =	9.18	Feet
Footing Width, BA =	19.68	Feet
Wall Length Considered =	1.00	ft

AREA #		Vertical: Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horizontal: Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
ES	$P_{con}(h)$ = p*H*Length =				2.51	15.68	39.34
	$P_{con}(v)$ = q*BE*Length =	2.30	15.09	34.63			
TOTAL ES		**2.30**		**34.63**	**2.51**		**39.34**

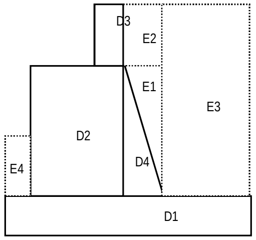
CANTILEVER ABUTMENT DESIGN - PRIMARY LOADS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	ALH
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Abutment	**Date:**	June 25, 2014

Calculate Live Loads

Primary Loads Section : 3.0



Superstructure Loads: Vertical: Horizontal:

AREA #		Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
LL+IM+PL	**Superstructure**	**5.13**	**7.71**	**39.54**			
BR	**Superstructure**				**0.501**	**29.20**	**14.63**

* See the load column under "Additional Loads on Structure" in the "General Loading Parameters" section for the above forces.

Live Load Surcharge Loads: LS

Per AASHTO 3.11.6.4, a live load surcharge shall be applied where vehicular load is expected to act on the surface of the backfill within a distance equal to one-half the wall height behind the back face of the wall.
If the surcharge is for highway, the intensity of the load shall be consistent with provisions of Article 3.6.1.2. See Tables 3.11.6.4-1 and 3.11.6.4-2 for equivalent heights.

Compute Horizontal Live Load Surcharge: (To be used for bearing pressure and sliding load cases):

Ke =	0.320	
Unit Weight of Soil, γ =	130.000	pcf
Surcharge Height, heq =	2.00	Feet
LS(h) =(Ke)(γ)(heq)*H =	2.61	kips
Moment arm = H/2 =	15.68	kips

Compute Vertical Live Load Surcharge: (To be used for bearing pressure cases only):

LS(v) =(γ)(heq)(BD+BE) =	2.39	kips
Moment arm = Ba-(BD+BE)/2 =	15.09	kips

Compute Vertical Live Load Surcharge: (To be used for heel reinf cases only):

LS(v) =(γ)(heq)(BE) =	2.39	kips
Moment arm (to back of batter) = BE/2 =	4.59	kips

Live Load Surcharge, LS: Summary Vertical: Horizontal:

AREA #		Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
LS	**LS(v)**	**2.39**	**15.09**	**36.02**			
	LS(h)				**2.61**	**15.68**	**40.91**

Total Live Load Load: Vertical: Horizontal:

AREA #	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
TOTAL LL+IM+PED+BR+LS	**7.51**		**75.56**	**3.11**		**55.54**
TOTAL LL+IM+PED+BR+LS (Sliding Only)	**5.13**		**39.54**	**3.11**		**55.54**
TOTAL LS (Heel Reinf Only)	**2.39**	**4.59**	**10.96**			

CANTILEVER ABUTMENT DESIGN - PRIMARY LOADS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	ALH
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Abutment	**Date:**	June 25, 2014

Calculate Water load (Buoyancy Forces)

Primary Loads Section : 4.0

HEIGHT OF STEM AT HIGH WATER:	19.68	
HEIGHT OF FOOTING AT HIGH WATER:	3.94	
WIDTH OF FOOTING, BA	19.68	
SOIL WEIGHT - WATER WEIGHT	67.60	pcf
UPWARD BOUYANT FORCE	-62.40	pcf

Horizontal Force = B(h) = (γ-(γ-62.4))*Ka)H^2/2, acts at HD/3:

INCLUDE HORIZONTAL FORCE? N

<-- Note: The Horizontal load is Not Applicable since the hydrostatic force is equal and opposite on both sides.

Bouyant Load, WA:

AREA #		VOLUME (CF)	GAMMA (#/CF)	Vertical: Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horizontal: Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
WA	B1 (Ftg)	77.46	-62.40	-4.83	9.84	-47.56			
	B2 (Stem)	77.46	-62.40	-4.83	8.53	-41.24			
	B3 (Soil over Ftg)	309.84	-62.40	-19.33	15.09	-291.75			
	STATIC						5.57	7.87	43.83
	SEISMIC						12.72	7.87	100.14
TOTAL WA (BL) (Static)				**-29.00**		**-380.55**	**0.00**		**0.00**
TOTAL WA (BL) (Seismic)				**-29.00**		**-380.55**	**0.00**		**0.00**

Calculate Stream Flow Pressure

Primary Loads Section : 4.1

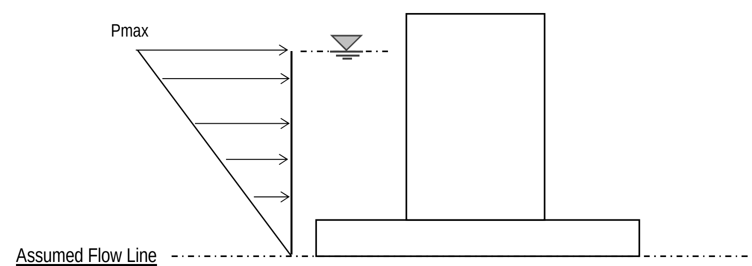
Note: The flow line is conservatively assumed to act at the bottom of the footing

Pmax:	0.0000	ksf
APPLIED:	N	

Force = 0.5 * Pmax * HD

Arm = HD * (2/3)

LOAD	HORIZONTAL: **FORCE** **(Kips)**	**ARM** **(Feet)**	**MOM** **(Ft x K)**
WA (SF)	0.00	15.74	0.00



Calculate Water Load & Stream Flow Load WA

Primary Loads Section : 4.2

Water Load (Bouyancy) & Stream Flow, WA:

AREA #	VOLUME (CF)	GAMMA (#/CF)	Vertical: Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horizontal: Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
TOTAL WA (Static)			-29.00		-380.55	0.00		0.00
TOTAL WA (Seismic)			-29.00		-380.55	0.00		0.00

CANTILEVER ABUTMENT DESIGN - PRIMARY LOADS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	ALH
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Abutment	**Date:**	June 25, 2014

Calculate Wind Loads

Primary Loads Section : 5.0

Superstructure Loads: Vertical: Horizontal:

AREA #		Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
WS	**Superstructure**				**0.00**	**29.20**	**0.00**
WL	**Superstructure**				**0.00**	**29.20**	**0.00**

* See the load column under "Additional Loads on Structure" in the "General Loading Parameters" section for the above forces.

Calculate Temperature Loads

Primary Loads Section : 6.0

Superstructure Loads: Vertical: Horizontal:

AREA #		Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
TU	**Superstructure**				**0.00**	**29.20**	**0.00**

* See the load column under "Additional Loads on Structure" in the "General Loading Parameters" section for the above forces.

CANTILEVER ABUTMENT DESIGN - PRIMARY LOADS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	ALH
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Abutment	**Date:**	June 25, 2014

Calculate Seismic Forces

Primary Loads Section : 7.0

Superstructure Loads: Vertical: Horizontal:

AREA #		Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
EQ	**Superstructure**				**6.161**	**29.20**	**179.87**

* See the load column under "Additional Loads on Structure" in the "General Loading Parameters" section for the above forces.

Substructure Loads:

(Ref: AASHTO 4th Ed., A11.1.1.1 for Mononobe-Okabe Analysis.)

GAMMA = unit weight of soil =	130.00	Lbs/CF		
H = height of soil face =	31.36	Feet		
PHI = angle of internal friction of soil =	33.00	Degrees =	0.58	Radians
DELTA = angle of friction between soil & abut =	22.00	Degrees =	0.38	Radians
i = backfill slope angle =	0.00	Degrees =	0.00	Radians
BETA = slope of wall to the vertical	0.00	Degrees =	0.00	Radians

A =	0.29			
kh = horizontal acceleration coefficient	0.435	Consider Cohesion?	N	------> kh = a * 0.5, Wall is NOT Restrained from Horizontal Movement
kv = vertical acceleration coefficient	0.000			
THETA = arc tan (kh/(1-Kv)) =	23.51	Degrees =	0.41	Radians
Kae (per AASHTO Eq. A11.1.1.1-2) =	**0.731**	**<===== Governs.**		

Earth Pressure Coefficients to be Used for Design per Geotechnical Report:

Kae (geotech) = **0.000** **<=== Does not govern.**

N/A
NOT GIVEN IN GEOTECH
REPORT

Load inclination from horizontal = δ =	22.00	degrees
Lateral EQ Load, Eae = 1/2*γ*Kae*H^2*(1-kv) =	46.73	klf
Arm for Horiz Load above BOF = H/3 =	10.45	ft (AASHTO pg 11-112)
Arm for Vert Load from Toe = BA =	19.68	ft

Consider for Sliding, Overturning, Bearing Pressure and Footing Reinforcement:

Vertical Component, Eav = Eae*sin(δ) =	17.51	klf	Include EQ In Design =	Y
Horizontal Component, Eah = Eae*cos(δ) =	43.33	klf	EQ Factor =	1

CANTILEVER ABUTMENT DESIGN - PRIMARY LOADS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	ALH
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Abutment	**Date:**	June 25, 2014

Calculate Seismic Forces

Primary Loads Section : 7.1

Include Seismic Passive Earth Pressure	Y	
Epe Factor	1	
kh = horizontal acceleration coefficient	0.435	
∅ = Soil Friction Angle	33.00	degrees
δ = Wall Interface Friction	22.00	degrees = 2/3 * ∅ --> 11.6.5.5
Kpe = Seismic Passive Earth Pressure Coefficient	3.13	Fig A11.4-2
γ = Unit Weight of Soil	130.00	pcf
Hff = Height of Soil at Front Face -1'	13.80	ft
Lateral EQ Load, Epe = 1/2*γ*Kpe*H^2=	38.72	klf ---> Equation A11.4-4
Horizontal Component, Eah (calculated earlier) =	43.33	klf > Kpe Calculated Above
=====> Use Epe =	38.72	klf
Arm for Horiz Load above BOF = Hff/3 =	4.60	ft (AASHTO pg 11-112)

SECTION 11: WALLS, ABUTMENTS, AND PIERS 11-117

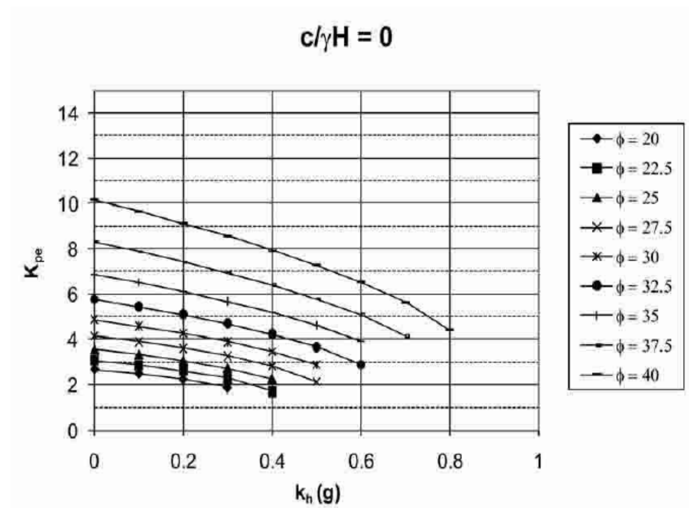


Figure A11.4-2—Seismic Passive Earth Pressure Coefficient Based on Log Spiral Procedure for $c/\gamma H$ = 0 and 0.05 (c = soil cohesion, γ = soil unit weight, and H = height or depth of wall over which the passive resistance acts)

Note: $k_h = A_s = k_{ho}$ for wall heights greater than 20 ft.

CANTILEVER ABUTMENT DESIGN - PRIMARY LOADS

General Information

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Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Abutment	**Date:**	June 25, 2014

Calculate Seismic Forces Continued..

Primary Loads Section : 7.2

WALL INERTIA EFFECTS

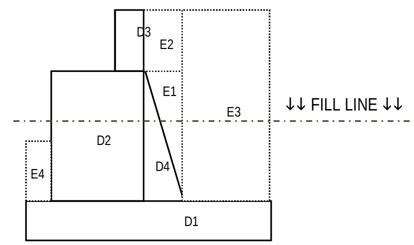
Per **AASHTO DIV 1A 6.4.3**, seismic design should take into account forces arising from seismically inducd lateral earth pressures (as computed above), additional forces arising from wall inertia and the transfer of seismic forces from the bridge deck through bearing supports which do not slide freely.

The following table computes the inertia forces due to the weight of the concrete and backfill.

kh = 0.435

FOR PIERS: Include DL above Fill Only

AREA #		DL (Kips)	DL*kh (Kips)	ARM (Feet)	MOM (Ft x K)	% of DL to be included
DL Wall	D1	11.62	5.05	1.97	9.95	100%
	D2	14.91	6.49	16.57	107.47	43%
	D3	0.53	0.23	30.28	7.01	100%
	D4	0.00	0.00	12.36	0.00	100% n/a
	Subtotal	**27.07**	**11.77**	**10.57**	**124.43**	
DL Backfill	E1	0.00	0.00	20.78	0.00	100%
	E2	0.00	0.00	30.28	0.00	100%
	E3	147.00	63.95	17.65	1128.53	100%
	E4	9.27	4.03	9.37	37.76	100%
	Subtotal	**156.27**	**67.98**	**17.16**	**1166.28**	
TOTAL		**183.33**	**79.75**	**16.18**	**1290.71**	



Total Seismic Loads, EQ:

AREA #		Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
EQ	EQ Superstructure =				6.161	29.20	179.868
	Eae(v)	17.51	19.68	344.51			
	Eae(h)				43.33	10.45	452.92
	Epe(v)		19.68	0.00			
	Epe				-38.72	4.60	-178.07
	Fwi(h)				79.75	16.18	1290.71
TOTAL EQ		**17.51**		**344.51**	**90.52**		**1745.43**

% Eae(h) to be included:

100% **FOR PIERS: M-O ANALYSIS IS FOR RETAINED SOILS --> N/A FOR PIERS**

CANTILEVER ABUTMENT DESIGN - PRIMARY LOADS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	ALH
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Abutment	**Date:**	June 25, 2014

Calculate Vehicle Collision Loads

Primary Loads Section : 8.2

Superstructure Loads: Vertical: Horizontal:

AREA #		Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
CT (Stem Design)	**Superstructure**				**0.00**	**0.00**	**0.00**
CT	**Superstructure**				**0.00**	**0.00**	**0.00**

* See the load column under "Additional Loads on Structure" in the "General Loading Parameters" section for the above forces.

Summary of Primary Loads

Primary Loads Section : 9.2

		Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
TOTAL DC (Super + Sub)		34.69		305.51			
TOTAL DW (Super)		0.45		3.49			
TOTAL DC (Substr. Only - Construction)		27.07		246.73			
Controlling Earth Pressures		0.87	19.68	17.10	2.50	13.32	33.31
0.00		1.71	19.68	33.57	4.22	4.93	20.82
TOTAL EV		41.99		524.28			
TOTAL ES		2.30		34.63	2.51		39.34
TOTAL LL+IM+PED+BR+LS		7.51	0.00	75.56	3.11	0.00	55.54
TOTAL LL+IM+PED+BR+LS (Sliding Only)		5.13	0.00	39.54	3.11	0.00	55.54
TOTAL LS (Heel Reinf Only)		2.39	4.59	10.96	0.00	0.00	0.00
TOTAL WA (Static)		-29.00		-380.55	0.00		0.00
TOTAL WA (Seismic)		-29.00		-380.55	0.00		0.00
WS	Superstructure				0.00	29.20	0.00
WL	Superstructure				0.00	29.20	0.00
TU	Superstructure				0.00	29.20	0.00
TOTAL EQ		17.51		344.51	90.52		1745.43
CT (Stem Design)		0.00	0.00	0.00	0.00	0.00	0.00
CT		0.00	0.00	0.00	0.00	0.00	0.00

CANTILEVER ABUTMENT DESIGN - LOAD COMBINATIONS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	ALH
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Abutment	**Date:**	June 25, 2014

References:

- AASHTO LRFD Bridge Design Specifications, 6th Edition, 2012
- ACI 318-08 Building Code Requirements for Structural Concrete, 2005
- 2009 MassDOT LRFD Bridge Manual, including draft Novemeber 2012 provisions
- AASHTO Guide Specifications for LRFD Seismic Bridge Design 2011

Notes:

This template assumes that the soils strata behind the abutment is uniform (only 1 strata is considered).

Khost Bridge Notes

Summary of Primary Loads

Load Combinations : 1.0

INCLUDE SEISMIC = Y

Load		Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)	Notes	LRFD Load Combination Load Case
Dead Load	$DC_{SUB+SUPER}$	34.69	0.00	305.51	0.00	0.00	0.00	Super + Sub	
	DW	0.45	0.00	3.49	0.00	0.00	0.00	Super Only	
	DC_{SUB}	27.07	0.00	246.73	0.00	0.00	0.00	Sub Only - Construction	LC1 only
Earth Load	EH	0.87	19.68	17.10	2.50	13.32	33.31	All cases except Heel	Used in all load cases
	EH	1.71	19.68	33.57	4.22	4.93	20.82	For Heel Reinforcement	Not used in any load case
	EV	41.99	0.00	524.28	0.00	0.00	0.00		
Earth Load Surcharge	ES	2.30	0.00	34.63	2.51	0.00	39.34		
Live Load Surcharge	LS(v)	2.39	15.09	36.02	0.00	0.00	0.00		
	LS(h)	0.00	0.00	0.00	2.61	15.68	40.91		
Live Load	LL+IM+PED+BR+LS	7.51	0.00	75.56	3.11	0.00	55.54		
	LL+IM+PED+BR+LS	5.13	0.00	39.54	3.11	0.00	55.54	No LS for Sliding LC	LC4, LC8 & LC10
	LS	2.39	4.59	10.96	0.00	0.00	0.00		
Bouyant Load & Stream Force	WA	-29.00	0.00	-380.55	0.00	0.00	0.00	Static	
	WA	-29.00	0.00	-380.55	0.00	0.00	0.00	Seismic	LC9 &LC10
Wind Load	WS	0.00	0.00	0.00	0.00	29.20	0.00		
	WL	0.00	0.00	0.00	0.00	29.20	0.00		
Temperature Load	TU	0.00	0.00	0.00	0.00	29.20	0.00		
Seismic Load	EQ	17.51	0.00	344.51	90.52	0.00	1745.43		
Vehicle Collision Load	CT	0.00	0.00	0.00	0.00	0.00	0.00	Stem Wall	LC11 & LC12
	CT	0.00	0.00	0.00	0.00	0.00	0.00	Stability	

CANTILEVER ABUTMENT DESIGN - LOAD COMBINATIONS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	ALH
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Abutment	**Date:**	June 25, 2014

Limit States and Load Factors

Load Combinations : 2.0

Service Limit State

Per AASHTO 10.5.2, foundation design at the service limit state shall include settlements, horizontal movements, overall stability (of earth slopes) and scour at the design flood.
* These items are part of the geotechnical scope and are therefore NOT included in this design.

Strength Limit States

Per AASHTO 10.5.3, foundation design at the strength limit strength shall include structural resistance, scour, nominal bearing resistance, overturning or excessive loss of contact, sliding and constructability.
* These items, except scour, are addressed in this design.

Extreme Events Limit States

Per AASHTO 10.5.4, foundation shall be designed for extreme events such as a seismic event and vehicle collision.
* These items are addressed in this design.

Since these factors are 1.0, they have not yet been incorporated into the design template.

Computation of the Load Modification Factor, h_i:

		Extreme	Strength
h_D Ductility Factor, (AASHTO 1.3.3):		1.00	1.00
h_R Redundancy Factor, (AASHTO 1.3.4):		1.00	1.00
h_I Operational Importance Factor, (AASHTO 1.3.5):		1.00	1.00
h_i (for loads for which γ_i(max) is appropriate) (AASHTO Eq 1.3.2.1-2):	$h_i = h_D h_R h_I \geq 0.95$	1.00	1.00
h_i (for loads for which γ_i(min) is appropriate) (AASHTO Eq 1.3.2.1-3):	$h_i = 1 / h_D h_R h_I \leq 1.00$	1.00	1.00

h_D Ductility Factor (for all other limit states h_D = 1.00)

$h_D \geq$	1.05	for nonductile components and connections.
$h_D =$	1.00	for conventional designs and details complying with the specifications.
$h_D \geq$	0.95	for components and connections for which additional ductility-enhancing m

h_R Redundancy Factor (for all other limit states h_R = 1.00)

$h_R \geq$	1.05	for nonredundant members
$h_R =$	1.00	for conventional levels of redundancy
$h_R \geq$	0.95	for exceptinal levels of redundancy

h_I Operational Importance Factor

$h_I \geq$	1.05	for a bridge of operational importance
$h_I =$	1.00	for typical bridges
$h_I \geq$	0.95	for relatively less important bridges

Load Factors for Permanent Loads (per AASHTO Table 3.4.1-2), g_p:

	Maximum	Minimum	
DC (Dead Load, General):	1.25	0.90	
DW (Wearing Surface & Utilities):	1.50	0.65	
EH (Horiz Earth):	1.43	0.90	<-- An average of Active and At-rest Coefficients used based on MHD's earth pressure design guidelines.
ES (Horiz Earth):	1.50	0.75	
EV (Vertical Earth, Retaining Structure):	1.35	1.00	

Live Load Factor During a Seismic Event, g_{EQ}:

	Maximum	Minimum	
g_{EQ} (AASHTO C3.4.1):	0.50	0.00	<--- Seismic Included

CANTILEVER ABUTMENT DESIGN - LOAD COMBINATIONS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	ALH
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Abutment	**Date:**	June 25, 2014

LRFD Load Combinations & Notes

Load Combinations : 3.0

NOTES:

1. Load Combination Strength II does not need to be checked since it applies to special design vehicles.
2. Load Combination Strength III does not need to be checked during construction since WS is not a significant load.
3. Load Combination Strength IV does not need to be checked since it applies to bridges with very high dead load to live load ratios.
4. Load Combination Strength V does not need to be checked during construction since WS and WL are not significant loads.
5. Extreme Event load combinations do not need to be checked during construction.
6. Extreme Event II load combinations does not need to be checked for abutments.
7. Service limit state load combinations do not need to be checked for abutment stability / reinforcement.
8. Fatigue limit state load combinations do not need to be checked for abutment stability / reinforcement.
9. All remaining load cases shall be checked using load factors which would provide max effect for either bearing or sliding / eccentricity similar to AASHTO Figures C11.5.5-1 and C11.5.5.2.
10. Bouyancy has been included in sliding load combinations. A load factor of 0.0 has been used for bearing pressure load combinations since it is conservative to ignore sliding for these computations.

Strength	LC1	LC1 - STRENGTH I CONSTRUCTION (Before Bridge Construction): gp max*(DCsub)+gp max*(EH)+gp max*(EV)+γp max*(ES)
Strength	LC2	LC2 - STRENGTH I CONSTRUCTION (Before Bridge LL): gp max*(DC+DW)+gp max*(EH)+gp max*(EV)+γp max*(ES)
Bearing	LC3	LC3 - STRENGTH I BEARING: gp max*(DC+DW)+gp max*(EH)+gp max*(EV)+1.75*(LL+IM+PL+BR+LS)+1.0*(WA)+0.50*(TU)
Sliding	LC4	LC4 - STRENGTH I SLIDING: gp min*(DC+DW)+gp max*(EH)+gp min*(EV)+1.75*(LL+IM+PL+BR+LS)+1.0*(WA)+0.50*(TU)
Bearing	LC5	LC5 - STRENGTH III BEARING: gp max*(DC+DW)+gp max*(EH)+gp max*(EV)+1.0*(WA)+1.4*(WS)+0.50*(TU)
Sliding	LC6	LC6 - STRENGTH III SLIDING: gp min*(DC+DW)+gp max*(EH)+gp min*(EV)+1.0*(WA)+1.4*(WS)+0.50*(TU)
Bearing	LC7	LC7 - STRENGTH V BEARING: gp max*(DC+DW)+gp max*(EH)+gp max*(EV)+1.35*(LL+IM+PL+BR+LS)+1.0*(WA)+0.4*(WS)+1.0*(WL)+0.50*(TU)
Sliding	LC8	LC8 - STRENGTH V SLIDING: gp min*(DC+DW)+gp max*(EH)+gp min*(EV)+1.35*(LL+IM+PL+BR+LS)+1.0*(WA)+0.4*(WS)+1.0*(WL)+0.50*(TU)
Extreme Bearing	LC9	LC9 - EXTREME EVENT I BEARING: gp max*(DC+DW)+gp max*(EH)+gp max*(EV)+gEQ MAX*(LL+IM+PL+BR+LS)+1.0*(EQ)
Extreme Sliding	LC10	LC10 - EXTREME EVENT I SLIDING: gp min*(DC+DW)+gp max*(EH)+gp min*(EV)+gEQ MIN*(LL+IM+PL+BR+LS)+1.0*(WA)+1.0*(EQ)
Extreme Bearing	LC11	LC11 - EXTREME EVENT II BEARING: gp max*(DC+DW)+gp max*(EH)+gp max*(EV)+0.50*(LL+IM+PL+BR+LS)+1.0*(CT)
Extreme Sliding	LC12	LC12 - EXTREME EVENT II SLIDING: gp min*(DC+DW)+gp max*(EH)+gp min*(EV)+0.50*(LL+IM+PL+BR+LS)+1.0*(WA)+1.0*(CT)

CANTILEVER ABUTMENT DESIGN - LOAD COMBINATIONS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	ALH
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Abutment	**Date:**	June 25, 2014

LRFD Load Combinations

Load Combinations : 3.1

↓ N/A, Valid for Pile Design Only ↓

NA (for Bottom row of piles) From Pile Design = 0

Bottom Row to Edge of Toe = 0

↓ N/A, Valid for Pile Design Only ↓

Distance of Pile Group N.A. From Footing Toe (See Pile Design Spreadsheet): 0.00 ft

LC1 - STRENGTH I CONSTRUCTION (Before Bridge Construction): $g_{p\,max}$*(DCsub)+$g_{p\,max}$*(EH)+$g_{p\,max}$*(EV)+$\gamma_{p\,max}$*(ES)

LOAD	Load Factor	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC_{SUB}	1.25	33.83		308.41	0.00		0.00
EH	1.43	1.24		24.37	3.56		47.46
EV	1.35	56.69		707.77	0.00		0.00
ES	1.50	3.44		51.95	3.76		59.01
SUM		**95.21**		**1092.50**	**7.33**		**106.47**

Distance of Vertical Force (V) From The Footing Toe	Offset of Pile Group N.A. From Original Location of V	Equivalent Moment Due to Offset of Pile Group N.A. From Original Location of V	**Mom. to Be Used On Pile Group = O.T. Mom. - Equivalent Mom.**	**Vertical Force to Be Used On Pile Group**	**Horizontal Force to Be Used On Pile Group**
11.48 ft	11.48 ft	1092.5 k.ft	**-986.0 k.ft**	**95.2 kip**	**7.3 kip**

LC2 - STRENGTH I CONSTRUCTION (Before Bridge LL): $g_{p\,max}$*(DC+DW)+$g_{p\,max}$*(EH)+$g_{p\,max}$*(EV)+$\gamma_{p\,max}$*(ES)

LOAD	Load Factor	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	1.25	43.36		381.89	0.00		0.00
DW	1.5	0.68		5.23	0.00		0.00
EH	1.43	1.24		24.37	3.56		47.46
EV	1.35	56.69		707.77	0.00		0.00
ES	1.50	3.44		51.95	3.76		59.01
SUM		**105.41**		**1171.21**	**7.33**		**106.47**

↓ N/A, Valid for Pile Design Only ↓

11.11 ft	11.11 ft	1171.2 k.ft	**-1064.7 k.ft**	**105.4 kip**	**7.3 kip**

CANTILEVER ABUTMENT DESIGN - LOAD COMBINATIONS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	ALH
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Abutment	**Date:**	June 25, 2014

LRFD Load Combinations Cont.

Load Combinations : 3.2

LC3 - STRENGTH I BEARING: $g_{p\ max}$*(DC+DW)+$g_{p\ max}$*(EH)+$g_{p\ max}$*(EV)+1.75*(LL+IM+PL+BR+LS)+1.0*(WA)+0.50*(TU)

LOAD	Load Factor	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	1.25	43.36		381.89	0.00		0.00
DW	1.5	0.68		5.23	0.00		0.00
EH	1.43	1.24		24.37	3.56		47.46
EV	1.35	56.69		707.77	0.00		0.00
LL+IM+PL+BR+LS	1.75	13.15		132.23	5.44		97.20
WA	1.00	-29.00		-380.55	0.00		0.00
TU	0.50	0.00		0.000	0.0000		0.000
SUM		**86.12**		**870.94**	**9.01**		**144.66**

Load Factors Based on this particular LRFD Combination

↓ N/A, Valid for Pile Design Only ↓

10.11 ft	10.11 ft	870.9 k.ft	**-726.3 k.ft**	**86.1 kip**	**9.0 kip**

LC4 - STRENGTH I SLIDING: $g_{p\ min}$*(DC+DW)+$g_{p\ max}$*(EH)+$g_{p\ min}$*(EV)+1.75*(LL+IM+PL+BR+LS)+1.0*(WA)+0.50*(TU)

LOAD	Load Factor	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	0.9	31.22		274.96	0.00		0.00
DW	0.65	0.29		2.27	0.00		0.00
EH	1.43	1.24		24.37	3.56		47.46
EV	1.00	41.99		524.28	0.00		0.00
LL+IM+PL+BR+LS	1.75	8.97		69.20	5.44		97.20
WA (static)	1.00	-29.00		-380.55	0.00		0.00
TU	0.50	0.00		0.00	0.000		0.000
SUM		**54.72**		**514.52**	**9.01**		**144.66**

Load Factors Based on this particular LRFD Combination

↓ N/A, Valid for Pile Design Only ↓

9.40 ft	9.40 ft	514.5 k.ft	**-369.9 k.ft**	**54.7 kip**	**9.0 kip**

LC5 - STRENGTH III BEARING: $g_{p\ max}$*(DC+DW)+$g_{p\ max}$*(EH)+$g_{p\ max}$*(EV)+1.0*(WA)+1.4*(WS)+0.50*(TU)

LOAD	Load Factor	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	1.25	43.36		381.89	0.00		0.00
DW	1.5	0.68		5.23	0.00		0.00
EH	1.425	1.24		24.37	3.56		47.46
EV	1.35	56.69		707.77	0.00		0.00
WA (static)	1.00	-29.00		-380.55	0.00		0.00
WS	1.40	0.00		0.00	0.00		0.00
TU	0.50	0.00		0.00	0.0000		0.0000
SUM		**72.97**		**738.71**	**3.56**		**47.46**

Load Factors Based on this particular LRFD Combination

↓ N/A, Valid for Pile Design Only ↓

10.12 ft	10.12 ft	738.7 k.ft	**-691.2 k.ft**	**73.0 kip**	**3.6 kip**

CANTILEVER ABUTMENT DESIGN - LOAD COMBINATIONS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	ALH
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Abutment	**Date:**	June 25, 2014

LRFD Load Combinations Cont.

Load Combinations : 3.3

LC6 - STRENGTH III SLIDING: $g_{p\,min}$*(DC+DW)+$g_{p\,max}$*(EH)+$g_{p\,min}$*(EV)+1.0*(WA)+1.4*(WS)+0.50*(TU)

LOAD	Load Factor	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	0.90	31.22		274.96	0.00		0.00
DW	0.65	0.29		2.27	0.00		0.00
EH	1.43	1.24		24.37	3.56		47.46
EV	1.00	41.99		524.28	0.00		0.00
WA	1.00	-29.00		-380.55	0.00		0.00
WS	1.40	0.00		0.00	0.00		0.00
TU	0.50	0.00		0.00	0.0000		0.0000
SUM		**45.74**		**445.32**	**3.56**		**47.46**

Load Factors Based on this particular LRFD Combination

↓ **N/A, Valid for Pile Design Only** ↓

9.73 ft	9.73 ft	445.3 k.ft	**-397.9 k.ft**	**45.7 kip**	**3.6 kip**

LC7 - STRENGTH V BEARING: $g_{p\,max}$*(DC+DW)+$g_{p\,max}$*(EH)+$g_{p\,max}$*(EV)+1.35*(LL+IM+PL+BR+LS)+1.0*(WA)+0.4*(WS)+1.0*(WL)+0.50*(TU)

LOAD	Load Factor	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	1.25	43.36		381.89	0.00		0.00
DW	1.5	0.68		5.23	0.00		0.00
EH	1.43	1.24		24.37	3.56		47.46
EV	1.35	56.69		707.77	0.00		0.00
LL+IM+PL+BR+LS	1.35	10.14		102.01	4.20		74.98
WA	1.00	-29.00		-380.55	0.00		0.00
WS	0.40	0.00		0.00	0.00		0.00
WL	1.00	0.00		0.00	0.00		0.00
TU	0.50	0.00		0.00	0.0000		0.0000
SUM		**83.11**		**840.71**	**7.76**		**122.45**

Load Factors Based on this particular LRFD Combination

↓ **N/A, Valid for Pile Design Only** ↓

10.12 ft	10.12 ft	840.7 k.ft	**-718.3 k.ft**	**83.1 kip**	**7.8 kip**

LC8 - STRENGTH V SLIDING: $g_{p\,min}$*(DC+DW)+$g_{p\,max}$*(EH)+$g_{p\,min}$*(EV)+1.35*(LL+IM+PL+BR+LS)+1.0*(WA)+0.4*(WS)+1.0*(WL)+0.50*(TU)

LOAD	Load Factor	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	0.9	31.22		274.96	0.00		0.00
DW	0.65	0.29		2.27	0.00		0.00
EH	1.425	1.24		24.37	3.56		47.46
EV	1	41.99		524.28	0.00		0.00
LL+IM+PL+BR+LS	1.35	6.92		53.38	4.20		74.98
WA	1.00	-29.00		-380.55	0.00		0.00
WS	0.40	0.00		0.00	0.00		0.00
WL	1.00	0.00		0.00	0.00		0.00
TU	0.50	0.00		0.00	0.0000		0.0000
SUM		**52.67**		**498.70**	**7.76**		**122.45**

Load Factors Based on this particular LRFD Combination

↓ **N/A, Valid for Pile Design Only** ↓

9.47 ft	9.47 ft	498.7 k.ft	**-376.3 k.ft**	**52.7 kip**	**7.8 kip**

CANTILEVER ABUTMENT DESIGN - LOAD COMBINATIONS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	ALH
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Abutment	**Date:**	June 25, 2014

LRFD Load Combinations Cont.

Load Combinations : 3.4

LC9 - EXTREME EVENT I BEARING: $g_{p\,max}$*(DC+DW)+$g_{p\,max}$*(EH)+$g_{p\,max}$*(EV)+$g_{EQ\,MAX}$*(LL+IM+PL+BR+LS)+1.0*(EQ)

LOAD	Load Factor	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	1.25	43.36		381.89	0.00		0.00
DW	1.5	0.68		5.23	0.00		0.00
EH	0.00	0.00		0.00	0.00		0.00
EV	1.35	56.69		707.77	0.00		0.00
LL+IM+PL+BR+LS	0.50	3.76		37.78	1.56		27.77
WA	0.00	0.00		0.00	0.00		0.00
EQ	1.00	17.51		344.51	90.52		1745.43
SUM		**121.99**		**1477.18**	**92.07**		**1773.20**

Load Factors Based on this particular LRFD Combination

↓ N/A, Valid for Pile Design Only ↓

12.11 ft	12.11 ft	1477.2 k.ft	**296.0 k.ft**	**122.0 kip**	**92.1 kip**

LC10 - EXTREME EVENT I SLIDING: $g_{p\,min}$*(DC+DW)+$g_{p\,max}$*(EH)+$g_{p\,min}$*(EV)+$g_{EQ\,MIN}$*(LL+IM+PL+BR+LS)+1.0*(WA)+1.0*(EQ)

LOAD	Load Factor	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	0.9	31.22		274.96	0.00		0.00
DW	0.65	0.29		2.27	0.00		0.00
EH	0.00	0.00		0.00	0.00		0.00
EV	1.00	41.99		524.28	0.00		0.00
LL+IM+PL+BR+LS	0.00	0.00		0.00	0.00		0.00
WA (seismic)	1.00	-29.00		-380.55	0.00		0.00
EQ	1.00	17.51		344.51	90.52		1745.43
SUM		**62.01**		**765.46**	**90.52**		**1745.43**

Load Factors Based on this particular LRFD Combination

↓ N/A, Valid for Pile Design Only ↓

12.34 ft	12.34 ft	765.5 k.ft	**980.0 k.ft**	**62.0 kip**	**90.5 kip**

CANTILEVER ABUTMENT DESIGN - LOAD COMBINATIONS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	ALH
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Abutment	**Date:**	June 25, 2014

LRFD Load Combinations Cont.

Load Combinations : 3.4

LC11 - EXTREME EVENT II BEARING: $g_{p\ max}$*(DC+DW)+$g_{p\ max}$*(EH)+$g_{p\ max}$*(EV)+$g_{EQ\ MAX}$*(LL+IM+PL+BR+LS)+1.0*(EQ)

LOAD	Load Factor	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	1.25	43.36		381.89	0.00		0.00
DW	1.5	0.68		5.23	0.00		0.00
EH	1.43	1.24		24.37	3.56		47.46
EV	1.35	56.69		707.77	0.00		0.00
LL+IM+PL+BR+LS	0.50	2.56		0.00	1.56		27.77
WA	0.00	0.00		0.00	0.00		0.00
CT	1.00	5.13		0.00	0.00		0.00
SUM		**109.66**		**1119.26**	**5.12**		**75.24**

Load Factors Based on this particular LRFD Combination

↓ N/A, Valid for Pile Design Only ↓

10.21 ft	10.21 ft	1119.3 k.ft	**-1044.0 k.ft**	**109.7 kip**	**5.1 kip**

LC12 - EXTREME EVENT II SLIDING: $g_{p\ min}$*(DC+DW)+$g_{p\ max}$*(EH)+$g_{p\ min}$*(EV)+$g_{EQ\ MIN}$*(LL+IM+PL+BR+LS)+1.0*(WA)+1.0*(EQ)

LOAD	Load Factor	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	0.9	31.22		274.96	0.00		0.00
DW	0.65	0.29		2.27	0.00		0.00
EH	1.43	1.24		24.37	3.56		47.46
EV	1.00	41.99		524.28	0.00		0.00
LL+IM+PL+BR+LS	0.50	2.56		0.00	0.00		0.00
WA (seismic)	1.00	-29.00		-380.55	0.00		0.00
CT	1.00	5.13		0.00	0.00		0.00
SUM		**53.44**		**445.32**	**3.56**		**47.46**

Load Factors Based on this particular LRFD Combination

↓ N/A, Valid for Pile Design Only ↓

8.33 ft	8.33 ft	445.3 k.ft	**-397.9 k.ft**	**53.4 kip**	**3.6 kip**

CANTILEVER ABUTMENT DESIGN

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	ALH
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Abutment	**Date:**	June 25, 2014

References:

- AASHTO LRFD Bridge Design Specifications, 6th Edition, 2012
- ACI 318-08 Building Code Requirements for Structural Concrete, 2005
- 2009 MassDOT LRFD Bridge Manual, including draft Novemeber 2012 provisions
- AASHTO Guide Specifications for LRFD Seismic Bridge Design 2011

Notes: **This template assumes that the soils strata behind the abutment is uniform (only 1 strata is considered).**

Check Bearing Resistance (per AASHTO 11.6.3.2) -- ON SOIL

Stability : 1.0

If supported on soil, the vertical stress (σ_v) shall be calculated assuming a uniformly distributed pressure (V) over an effective base area (B-2e). AASHTO Fig 11.6.3.2-1 ----> qr / $\Phi\beta$ = q_n =

If supported on rock, the vertical stress (σ_v) shall be calculated assuming a linearly distributed pressure over an effective base area. AASHTO Fig 11.6.3.2-2 ----> qr / $\Phi\beta$ = q_n =

Nominal Bearing Resistance, qn: q_n = 17.78 ksf

Strength Bearing Resistance Factor, $\Phi\beta$ (AASHTO Table 10.5.5.2.2): 0.45 — qr = $\Phi\beta$ * q_n = 8.00 ksf

Extreme Event Bearing Resistance Factor, $\Phi\beta$ (AASHTO 10.5.5.3.3): 1.00 — qr = $\Phi\beta$ * q_n = 17.78 ksf

	LOAD COMBINATION	Vertical Force (Kips)	Resisting Moment (Ft x K)	Overturn Moment (Ft x K)	Mnet (Ft x K)	Eccentricity from Toe, et=Mnet/V (Ft)	Eccentricity from CL, e=B/2-et (Ft)	σ_v on soil (ksf)	$\sigma_{v\,max}$ on rock (ksf)	$\sigma_{v\,min}$ on rock (ksf)	$\sigma_v < \Phi\beta{*}q_n$	
Strength	**LC1**	95.21	1092.50	106.47	986.03	10.36	0.52	5.11	5.60	4.08	OK	
Strength	**LC2**	105.41	1171.21	106.47	1064.74	10.10	0.26	5.50	5.78	4.93	OK	
Bearing	**LC3**	86.12	870.94	144.66	726.27	8.43	1.41	5.11	6.25	2.50	OK	
Sliding	**LC4**	54.72	514.52	144.66	369.85	6.76	3.08	4.05	5.39	0.17	N/A	<--*N/A Sliding Combination
Bearing	**LC5**	72.97	738.71	47.46	691.24	9.47	0.37	3.85	4.12	3.29	OK	
Sliding	**LC6**	45.74	445.32	47.46	397.85	8.70	1.14	2.63	3.13	1.51	N/A	<--*N/A Sliding Combination
Bearing	**LC7**	83.11	840.71	122.45	718.27	8.64	1.20	4.81	5.77	2.68	OK	
Sliding	**LC8**	52.67	498.70	122.45	376.25	7.14	2.70	3.69	4.88	0.48	N/A	<--*N/A Sliding Combination
Ex. Bearing	**LC9**	121.99	1477.18	1773.20	-296.02	-2.43	12.27	**	**	**	NO GOOD	
Ex. Sliding	**LC10**	62.01	765.46	1745.43	-979.97	-15.80	25.64	**	**	**	N/A	<--*N/A Ex. Sliding Combination
Ex. Bearing	**LC11**	109.66	1119.26	75.24	1044.03	9.52	0.32	5.76	6.11	5.03	OK	
Ex. Sliding	**LC12**	53.44	445.32	47.46	397.85	7.45	2.39	3.59	4.70	0.73	N/A	<--*N/A Ex. Sliding Combination

*** Sliding Load Combinations are Not Applicable for checking the Bearing**

**** Eccentricity is such that the resultant vertical force falls outside the footing, hence bearing pressure cannot be calculated.**

CANTILEVER ABUTMENT DESIGN

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	ALH
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Abutment	**Date:**	June 25, 2014

Check Overturning (per AASHTO 11.6.3.3) -- ON SOIL

Stability : 2.0

e allowable (ftgs on soil): 6.56 ft
e allowable (ftgs on rock): 8.86 ft
If e < e allowable, Overturning is OK:

	LOAD COMBINATION	Eccentricty from CL, e=B/2-et (Ft)	Check Overturning	
Strength	**LC1**	0.52	OK	
Strength	**LC2**	0.26	OK	
Bearing	**LC3**	1.41	OK	
Sliding	**LC4**	3.08	N/A	**<--*N/A Sliding Combination**
Bearing	**LC5**	0.37	OK	
Sliding	**LC6**	1.14	N/A	**<--*N/A Sliding Combination**
Bearing	**LC7**	1.20	OK	
Sliding	**LC8**	2.70	N/A	**<--*N/A Sliding Combination**
Ex. Bearing	**LC9**	12.27	NO GOOD	
Ex. Sliding	**LC10**	25.64	N/A	**<--*N/A Ex. Sliding Combination**
Ex. Bearing	**LC11**	0.32	OK	
Ex. Sliding	**LC12**	2.39	N/A	**<--*N/A Ex. Sliding Combination**

*** Sliding Load Combinations are Not Applicable for checking Overturning**

CANTILEVER ABUTMENT DESIGN

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	ALH
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Abutment	**Date:**	June 25, 2014

Check Sliding (per AASHTO 10.6.3.4)

Stability : 3.0

Ignore Passive Resistance of Soil per MassHighway

Strength Sliding Resistance Factor, Φ_τ (AASHTO Table 11.5.7-1):	1.00	
Extreme Event Sliding Resistance Factor, Φ_ϖ (AASHTO 10.5.5.3.3):	1.00	
Internal Friction Angle of Drained Soil, Φ_f:	33.00	degrees
tan δ: = tan Φ_f (per AASHTO 10.6.3.4-2):	0.65	for concrete against soil. Multiply by 0.8 for precast concrete footing

	LOAD COMBINATION	Vertical Force (Kips)	Rt = V * tan δ: (Kips)	Φ_τ (Strength) Φ_ω (Extreme) (Kips)	Nom. Sliding Resistance Φ_τ*Rt (Kips)	Horiz Force (Kips)	Check Sliding	
Strength	**LC1**	95.21	61.83	1.00	61.83	7.33	N/A	<--*N/A Strength Combination
Strength	**LC2**	105.41	68.45	1.00	68.45	7.33	N/A	<--*N/A Strength Combination
Bearing	**LC3**	86.12	55.93	1.00	55.93	9.01	N/A	<--*N/A Bearing Combination
Sliding	**LC4**	54.72	35.53	1.00	35.53	9.01	OK	
Bearing	**LC5**	72.97	47.39	1.00	47.39	3.56	N/A	<--*N/A Bearing Combination
Sliding	**LC6**	45.74	29.71	1.00	29.71	3.56	OK	
Bearing	**LC7**	83.11	53.97	1.00	53.97	7.76	N/A	<--*N/A Bearing Combination
Sliding	**LC8**	52.67	34.20	1.00	34.20	7.76	OK	
Ex. Bearing	**LC9**	121.99	79.22	1.00	79.22	92.07	N/A	<--*N/A Ex. Bearing Combination
Ex. Sliding	**LC10**	62.01	40.27	1.00	40.27	90.52	NO GOOD	
Ex. Bearing	**LC11**	109.66	71.21	0.65	46.25	0.00	N/A	<--*N/A Ex. Bearing Combination
Ex. Sliding	**LC12**	53.44	34.70	0.65	22.54	0.00	OK	

Results Summary:

Stability : 4.0

STABILITY RESULTS:

LOAD COMBINATION:	BEARING RESISTANCE	OVERTURNING	SLIDING	
LC1	**OK**	**OK**	N/A	<== Construction
LC2	**OK**	**OK**	N/A	<== Construction
LC3	**OK**	**OK**	N/A	
LC4	N/A	N/A	**OK**	
LC5	**OK**	**OK**	N/A	
LC6	N/A	N/A	**OK**	
LC7	**OK**	**OK**	N/A	
LC8	N/A	N/A	**OK**	
LC9	**NO GOOD**	**NO GOOD**	N/A	
LC10	N/A	N/A	**NO GOOD**	
LC11	**OK**	**OK**	N/A	
LC12	N/A	N/A	**OK**	

BACKUP CALCULATIONS TO BE USED IN THE INPUT TAB

PIER LOADING CALCULATIONS - LIVE LOAD (LL), BREAKING FORCE (BR) & SEISMIC LOAD (EQ)

GENERAL INFORMATION

Project Number:	1298\127-1298-12001-LT0077	Designed By:	alh
Description:	Khost Bridge No. 10	Checked By:	SAM
Structure:	Pier	Date:	6/25/2014

References: AASHTO LRFD Bridge Design Specifications, 6th Edition, 2012

Notes: This spreadsheet computes the loads on an abutment, considering the spans left or right of the abutment is simply supported.

SUPERSTRUCTURE LOADING ON ABUTMENT - VERTICAL FORCES (CONT.)

Live Load, LL

Type of Truck: HL-	93	
Roadway Width =	26.24	ft
Lane Width =	12	ft
Roadway / Lane Width =	2.19	
Use --> No of Lanes =	2	
Multiple Presence Factor, m =	1	

Table 3.6.1.1.2-1—Multiple Presence Factors, *m*

Number of Loaded Lanes	Multiple Presence Factors, *m*
1	1.20
2	1.00
3	0.85
>3	0.65

Truck Loading:

Section 3.6.1.2.2

	Left/Right Span			
Span Length, L =	39.92	ft		
Dynamic Load Allowance, (IM) =	1.33			Section 3.6.2.1
Number of Lanes =	2			
Multiple Presence Factor, m =	1.00			Section 3.6.1.1.2
Vmax =	55.2	kips / Lane	<-- T3.3.1.2 Shear & End Reactions	
Vmax =	110.40	kips	<- Vmax * m * # of lanes	
Reaction, LL V =	110.40	kips		
Reaction, (LL+IM) V =	146.8	kips	<-- IM * V	
Total Reaction, Truck (LL) =	110.4	kips		
Total Reaction, Truck (LL+IM) =	146.8	kips		

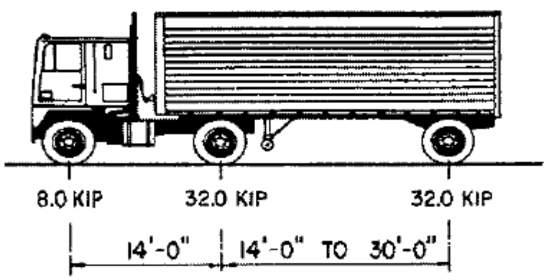


Figure 3.6.1.2.2-1

GENERAL INFORMATION

Project Number:	1298\127-1298-12001-LT0077	Designed By:	alh
Description:	Khost Bridge No. 10	Checked By:	SAM
Structure:	Pier	Date:	6/25/2014

SUPERSTRUCTURE LOADING ON ABUTMENT - VERTICAL FORCES (CONT.)

Tandem Loading: Section 3.6.1.2.3

	Left/Right Span			
L =	39.92	ft		
Dynamic Load Allowance, (IM) =	1.33			Section 3.6.2.1
Number of Lanes =	2			
Multiple Presence Factor, m =	1.00			Section 3.6.1.1.2
P1 =	25	kips		
P2 =	25	kips		
Axle Spacing =	4	ft		
Vmax =	47.49	kips/ Lane		
Vmax =	94.99	Kips	<- Vmax * m * # of lanes	
Reaction, LL V =	94.99	kips		
Reaction, (LL+IM) V =	126.34	kips	<-- IM * V	
Total Reaction, Tandem (LL) =	95.0	kips		
Total Reaction, Tandem (LL+IM) =	126.3	kips		

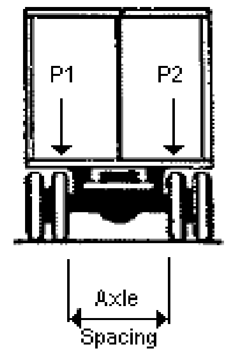
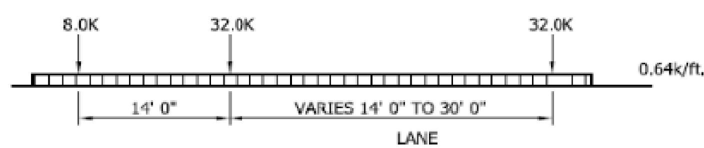


Figure 3.6.1.2.2-1

Live Load, LL (cont.)

Lane Loading: Section 3.6.1.2.4

	Left/Right Span			
L =	39.92	ft		
Number of Lanes =	2			
Multiple Presence Factor, m =	1.00			Section 3.6.1.1.2
Lane Load =	0.64	klf		
Vmax =	12.77	kips/ Lane		
Vmax =	25.55	Kips	<- Vmax * m * # of lanes	
Reaction, Lane Load (LL) =	25.5	kips		
Total Reaction, Lane Load (LL) =	25.5	kips		



BACKUP CALCULATIONS TO BE USED IN THE INPUT TAB

PIER LOADING CALCULATIONS - LIVE LOAD (LL), BREAKING FORCE (BR) & SEISMIC LOAD (EQ)

GENERAL INFORMATION

Project Number:	1298\127-1298-12001-LT0077	Designed By:	alh
Description:	Khost Bridge No. 10	Checked By:	SAM
Structure:	Pier	Date:	6/25/2014

SUPERSTRUCTURE LOADING ON ABUTMENT - VERTICAL FORCES (CONT.)

Pedestrian Live Load

Pedestrian Live Load, PL = 0.075 ksf <--- per AASHTO 3.6.1.6 for Sidewalks with a Width >= 2.0 ft

Width of Sidewalk = 3.94 ft

PL = 0.295 klf

Length of Sidewalk = 39.92 ft

PL = 11.78 kips

--> PL / Abutment = 5.89 kips

Bridge Width = 37.56 ft

--> PL / LF of Abutment = 0.16 klf / Sidewalk

No of Sidewalks = 2

--> PL / LF of Abutment = 0.31 klf

Live Loads

	LL	IM	LL + IM
Truck	55.20	1.33	73.42
Tandem	47.49	1.33	63.17
Lane	12.77	1	12.77
Truck + Lane	67.97		86.19
Tandem + lane	60.27		75.94
Max	67.97		86.19

Max = 86.19 kips

No of Lanes = 2.00

m = 1.00

LL+I = 172.38 kips

Abutment Length = 37.56 ft

LL+ I = 4.59 klf

LL + I + PL = 4.90 klf <-- INPUT LOAD

BACKUP CALCULATIONS TO BE USED IN THE INPUT TAB

PIER LOADING CALCULATIONS - LIVE LOAD (LL), BREAKING FORCE (BR) & SEISMIC LOAD (EQ)

GENERAL INFORMATION

Project Number:	1298\127-1298-12001-LT0077	Designed By:	alh
Description:	Khost Bridge No. 10	Checked By:	SAM
Structure:	Pier	Date:	6/25/2014

SUPERSTRUCTURE LOADING ON ABUTMENT - LATERAL FORCES

Braking Force, BR

Section 3.6.4

Notes: Dynamic Load Allowance increase not required. AASHTO3.6.2.1
Braking Force ONLY applies to fixed bearings
Braking Force includes multiple presence factor

Type of Bearing: Fixed

25% Axle Weight of Design Truck =	25%	18.00	kips	Design Truck Axle Weight =	72
25% Axle Weight of Design Tandem =	25%	12.50	kips	Design Tandem Axle Weight =	50
5% (Axle Weight of Design Truck + Lane Load) =	5%	4.88	kips	Design Truck + Lane Axle Weight =	97.55
5% (Axle Weight of Design Tandem Load + Lane Load) =	5%	3.78	kips	Design Tandem + Lane Axle Weight =	75.55

Braking Force on Abutment (BR) =	18	kips	<---- 25% Axle Weight of Design Truck
Number of Lanes =	2		
Multiple Presence Factor, m =	1		
BR =	0.96	klf	
No of Fixed Ends =	2		
BR =	0.48	klf	<--- BR / Abutment Length <-- Input Load

Location of Load Application = 0.00 ft above Bridge Seat

GENERAL INFORMATION

Project Number:	1298\127-1298-12001-LT0077	Designed By:	alh
Description:	Khost Bridge No. 10	Checked By:	SAM
Structure:	Pier	Date:	6/25/2014

SUPERSTRUCTURE LOADING ON ABUTMENT/PIER - LATERAL FORCES - EQ

Concrete Pryout (in Tension of a Single Anchor)

Nb = Kc * Sqrt (f'c) * $(hef)^{1.6}$

Kc =	24			
f'c =	3000	psi		
hef =	11.81	in		
Nb =	53351.51	lbs	53.35	kips
Anc =	675000	mm^2		
width =	900	mm		
Length =	750	mm		
Anco =	810000	mm^2		
width =	900	mm		
Length =	900	mm		
Anc / Anco =	0.83			

Ncb = (Anc / Anco) * ψed,N * ψc,N * ψcp,N * Nb

ψed,N =	0.90	= 0.7 + 0.3 (Ca,min / 1.5*hef)
Ca,min =	300	mm
1.5 * hef =	450.0	mm
ψc,N =	1.25	
ψcp,N =	1	
$\emptyset$ =	0.75	
Ncb =	50.02	kip / Anchor
$\emptyset$ Ncb =	37.51	kip / Anchor

BACKUP CALCULATIONS TO BE USED IN THE INPUT TAB

PIER LOADING CALCULATIONS - LIVE LOAD (LL), BREAKING FORCE (BR) & SEISMIC LOAD (EQ)

GENERAL INFORMATION

Project Number:	1298\127-1298-12001-LT0077	Designed By:	alh
Description:	Khost Bridge No. 10	Checked By:	SAM
Structure:	Pier	Date:	6/25/2014

SUPERSTRUCTURE LOADING ON ABUTMENT/PIER - LATERAL FORCES - EQ (CONT.)

Concrete Breakout (in shear of a single anchor)

$Vb = 7\ (le / do)^{0.2} * \mathrm{sqrt}\ (do) * \mathrm{sqrt}\ (f'c) * (Ca1)^{1.5}$ Ref: ACI 318 Eq (D-24)

do =	0.985	in	25.0	mm
hef =	11.81	in	300.0	mm
8 *do =	7.88	in	200.2	mm
le =	7.88	in	200.2	mm
Ca1 =	13.78	in	350.0	mm
Vb =	29503.16	lbs	29.50	kips

$Vcb = (Anc / Anco) * \psi ed,V * \psi c,V * Vb$

Anc / Anco =	1
ψed,V =	1
ψc,V=	1
$\varnothing$ =	0.75

Vcb =	29.50	kip / Anchor
$\varnothing$ Vcb =	22.13	kip / Anchor

	kips / Anchor	No of Anchors	Kips	
Concrete Pryout	37.51	10 in Tension	375.14	<-- 0 Anchors in Tension for Abutment
Concrete Breakout	22.13	10 in Shear	221.27	
Total EQ on Superstructure			596.41	Kips

GENERAL INFORMATION

Project Number:	1298\127-1298-12001-LT0077	Designed By:	alh
Description:	Khost Bridge No. 10	Checked By:	SAM
Structure:	Pier	Date:	6/25/2014

AASHTO Standard Specifications for Highway Bridges - 17th edition 2002

Loading -- HS 20-44 (MS18)

TABLE OF MAXIMUM MOMENTS, SHEARS, AND REACTIONS—SIMPLE SPANS, ONE LANE

Spans in feet; moments in thousands of foot-pounds; shears and reactions in thousands of pounds.

These values are subject to specification reduction for loading of multiple lanes.

Impact not included.

Span	Moment	End shear and end reaction (a)	Span	Moment	End shear and end reaction (a)
1	8.0(b)	32.0(b)	42	485.3(b)	56.0(b)
2	16.0(b)	32.0(b)	44	520.9(b)	56.7(b)
3	24.0(b)	32.0(b)	46	556.5(b)	57.3(b)
4	32.0(b)	32.0(b)	48	592.1(b)	58.0(b)
5	40.0(b)	32.0(b)	50	627.9(b)	58.5(b)
6	48.0(b)	32.0(b)	52	663.6(b)	59.1(b)
7	56.0(b)	32.0(b)	54	699.3(b)	59.6(b)
8	64.0(b)	32.0(b)	56	735.1(b)	60.0(b)
9	72.0(b)	32.0(b)	58	770.8(b)	60.4(b)
10	80.0(b)	32.0(b)	60	806.5(b)	60.8(b)
11	88.0(b)	32.0(b)	62	842.4(b)	61.2(b)
12	96.0(b)	32.0(b)	64	878.1(b)	61.5(b)
13	104.0(b)	32.0(b)	66	914.0(b)	61.9(b)
14	112.0(b)	32.0(b)	68	949.7(b)	62.1(b)
15	120.0(b)	34.1(b)	70	985.6(b)	62.4(b)
16	128.0(b)	36.0(b)	75	1,075.1(b)	63.1(b)
17	136.0(b)	37.7(b)	80	1,164.9(b)	63.6(b)
18	144.0(b)	39.1(b)	85	1,254.7(b)	64.1(b)
19	152.0(b)	40.4(b)	90	1,344.4(b)	64.5(b)
20	160.0(b)	41.6(b)	95	1,434.1(b)	64.9(b)
21	168.0(b)	42.7(b)	100	1,524.0(b)	65.3(b)
22	176.0(b)	43.6(b)	110	1,703.6(b)	65.9(b)
23	184.0(b)	44.5(b)	120	1,883.3(b)	66.4(b)
24	192.7(b)	45.3(b)	130	2,063.1(b)	67.6
25	207.4(b)	46.1(b)	140	2,242.8(b)	70.8
26	222.2(b)	46.8(b)	150	2,475.1	74.0
27	237.0(b)	47.4(b)	160	2,768.0	77.2
28	252.0(b)	48.0(b)	170	3,077.1	80.4
29	267.0(b)	48.8(b)	180	3,402.1	83.6
30	282.1(b)	49.6(b)	190	3,743.1	86.8
31	297.3(b)	50.3(b)	200	4,100.0	90.0
32	312.5(b)	51.0(b)	220	4,862.0	96.4
33	327.8(b)	51.6(b)	240	5,688.0	102.8
34	343.5(b)	52.2(b)	260	6,578.0	109.2
35	361.2(b)	52.8(b)	280	7,532.0	115.6
36	378.9(b)	53.3(b)	300	8,550.0	122.0
37	396.6(b)	53.8(b)			
38	414.3(b)	54.3(b)			
39	432.1(b)	54.8(b)			
40	449.8(b)	55.2(b)			

GENERAL INFORMATION

Project Number:	1298\127-1298-12001-LT0077	Designed By:	alh
Description:	Khost Bridge No. 10	Checked By:	SAM
Structure:	Pier	Date:	6/25/2014

Table 3.3.1.1
Maximum Unfactored HL-93 Live Load Moments, Shears, and Reactions
Simple Spans, One Lane, w/o Dynamic Load Allowance

SPAN	MOMENTS					SHEARS & END REACTIONS			
SPAN FT	TRUCK KIP-FT	TANDEM KIP-FT	LANE KIP-FT	TOTAL KIP-FT	SPAN PT. %	TRUCK KIP	TANDEM KIP	LANE KIP	TOTAL KIP
1	8.0	6.3	0.1	8.1	0.50	32.0	25.0	0.3	32.3
2	16.0	12.5	0.3	16.3	0.50	32.0	25.0	0.6	32.6
3	24.0	18.8	0.7	24.7	0.50	32.0	25.0	1.0	33.0
4	32.0	25.0	1.3	33.3	0.50	32.0	25.0	1.3	33.3
5	40.0	31.3	2.0	42.0	0.50	32.0	30.0	1.6	33.6
6	48.0	37.5	2.9	50.9	0.50	32.0	33.3	1.9	35.3
7	56.0	43.8	3.9	59.9	0.50	32.0	35.7	2.2	38.0
8	64.0	50.0	5.1	69.1	0.50	32.0	37.5	2.6	40.1
9	72.0	62.5	6.5	78.5	0.50	32.0	38.9	2.9	41.8
10	80.0	75.0	8.0	88.0	0.50	32.0	40.0	3.2	43.2
11	84.5	92.0	9.3	101.3	0.40	32.0	40.9	3.5	44.4
12	92.2	104.0	11.1	115.1	0.40	32.0	41.7	3.8	45.5
13	103.0	115.9	13.4	129.3	0.45	32.0	42.3	4.2	46.5
14	110.9	128.3	15.5	143.8	0.45	32.0	42.9	4.5	47.3
15	118.8	140.6	17.8	158.4	0.45	34.1	43.3	4.8	48.1
16	126.7	153.0	20.3	173.3	0.45	36.0	43.8	5.1	48.9
17	134.6	165.4	22.9	188.3	0.45	37.6	44.1	5.4	49.6
18	142.6	177.8	25.7	203.4	0.45	39.1	44.4	5.8	50.2
19	150.5	190.1	28.6	218.7	0.45	40.4	44.7	6.1	50.8
20	158.4	202.5	31.7	234.2	0.45	41.6	45.0	6.4	51.4
21	166.3	214.9	34.9	249.8	0.45	42.7	45.2	6.7	52.0
22	174.2	227.3	38.3	265.6	0.45	43.6	45.5	7.0	52.5
23	182.2	239.6	41.9	281.5	0.45	44.5	45.7	7.4	53.0
24	190.1	252.0	45.6	297.6	0.45	45.3	45.8	7.7	53.5
25	198.0	264.4	49.5	313.9	0.45	46.1	46.0	8.0	54.1
26	210.2	276.8	53.5	330.3	0.45	46.8	46.2	8.3	55.1
27	226.1	289.1	57.7	346.9	0.45	47.4	46.3	8.6	56.0
28	241.9	301.5	62.1	363.6	0.45	48.0	46.4	9.0	57.0
29	257.8	313.9	66.6	380.5	0.45	48.8	46.6	9.3	58.1
30	273.6	326.3	71.3	397.5	0.45	49.6	46.7	9.6	59.2
31	289.4	338.6	76.1	414.7	0.45	50.3	46.8	9.9	60.2
32	307.0	351.0	81.1	432.1	0.45	51.0	46.9	10.2	61.2
33	324.9	363.4	86.2	449.6	0.45	51.6	47.0	10.6	62.2
34	332.0	375.0	92.5	467.5	0.50	52.2	47.1	10.9	63.1
35	350.0	387.5	98.0	485.5	0.50	52.8	47.1	11.2	64.0
36	368.0	400.0	103.7	503.7	0.50	53.3	47.2	11.5	64.9
37	386.0	412.5	109.5	522.0	0.50	53.8	47.3	11.8	65.7
38	404.0	425.0	115.5	540.5	0.50	54.3	47.4	12.2	66.5
39	422.0	437.5	121.7	559.2	0.50	54.8	47.4	12.5	67.2
40	440.0	450.0	128.0	578.0	0.50	55.2	47.5	12.8	68.0

GENERAL INFORMATION

Project Number:	1298\127-1298-12001-LT0077	Designed By:	alh
Description:	Khost Bridge No. 10	Checked By:	SAM
Structure:	Pier	Date:	6/25/2014

LRFD BRIDGE DESIGN 3-9

Table 3.3.1.2

Maximum Unfactored HL-93 Live Load Moments, Shears, and Reactions
Simple Spans, One Lane, w/o Dynamic Load Allowance

	MOMENTS					SHEARS & END REACTIONS			
SPAN FT	TRUCK KIP-FT	TANDEM KIP-FT	LANE KIP-FT	TOTAL KIP-FT	SPAN PT. %	TRUCK KIP	TANDEM KIP	LANE KIP	TOTAL KIP
42	485.2	474.8	139.7	624.9	0.45	56.0	47.6	13.4	69.4
44	520.9	499.5	153.3	674.2	0.45	56.7	47.7	14.1	70.8
46	556.5	524.3	167.6	724.1	0.45	57.4	47.8	14.7	72.1
48	592.2	549.0	182.5	774.6	0.45	58.0	47.9	15.4	73.4
50	627.8	573.8	198.0	825.8	0.45	58.6	48.0	16.0	74.6
52	663.4	598.5	214.2	877.6	0.45	59.1	48.1	16.6	75.7
54	699.1	623.3	230.9	930.0	0.45	59.6	48.1	17.3	76.8
56	734.7	648.0	248.4	983.1	0.45	60.0	48.2	17.9	77.9
58	770.4	672.8	266.4	1036.8	0.45	60.4	48.3	18.6	79.0
60	806.0	697.5	285.1	1091.1	0.45	60.8	48.3	19.2	80.0
62	841.6	722.3	304.4	1146.1	0.45	61.2	48.4	19.8	81.0
64	877.3	747.0	324.4	1201.7	0.45	61.5	48.4	20.5	82.0
66	912.9	771.8	345.0	1257.9	0.45	61.8	48.5	21.1	82.9
68	948.6	796.5	366.2	1314.8	0.45	62.1	48.5	21.8	83.9
70	984.2	821.3	388.1	1372.3	0.45	62.4	48.6	22.4	84.8
75	1070.0	887.5	450.0	1520.0	0.50	63.0	48.7	24.0	87.0
80	1160.0	950.0	512.0	1672.0	0.50	63.6	48.8	25.6	89.2
85	1250.0	1012.5	578.0	1828.0	0.50	64.1	48.8	27.2	91.3
90	1340.0	1075.0	648.0	1988.0	0.50	64.5	48.9	28.8	93.3
95	1430.0	1137.5	722.0	2152.0	0.50	64.9	48.9	30.4	95.3
100	1520.0	1200.0	800.0	2320.0	0.50	65.3	49.0	32.0	97.3
110	1700.0	1325.0	968.0	2668.0	0.50	65.9	49.1	35.2	101.1
120	1880.0	1450.0	1152.0	3032.0	0.50	66.4	49.2	38.4	104.8
130	2060.0	1575.0	1352.0	3412.0	0.50	66.8	49.2	41.6	108.4
140	2240.0	1700.0	1568.0	3808.0	0.50	67.2	49.3	44.8	112.0
150	2420.0	1825.0	1800.0	4220.0	0.50	67.5	49.3	48.0	115.5
160	2600.0	1950.0	2048.0	4648.0	0.50	67.8	49.4	51.2	119.0
170	2780.0	2075.0	2312.0	5092.0	0.50	68.0	49.4	54.4	122.4
180	2960.0	2200.0	2592.0	5552.0	0.50	68.3	49.4	57.6	125.9
190	3140.0	2325.0	2888.0	6028.0	0.50	68.5	49.5	60.8	129.3
200	3320.0	2450.0	3200.0	6520.0	0.50	68.6	49.5	64.0	132.6

http://www.dot.nd.gov/manuals/bridge/lrfd-bridge-design/Section03A.pdf

PIER DESIGN -INPUT

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	ALH
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Pier	**Date:**	June 25, 2014

References:

AASHTO LRFD Bridge Design Specifications, 6th Edition, 2012	AASHTO Guide Specifications for LRFD Seismic Bridge Design 2011
ACI 318-08 Building Code Requirements for Structural Concrete, 2005	
2009 MassDOT LRFD Bridge Manual, including draft Novemeber 2012 provisions	

General Notes: **This template assumes that the soils strata behind the abutment is uniform (only 1 strata is considered).**

BRIDGE Design **Khost Bridge Notes**

Project Notes:

General Design Parameters

Input Section : 1.0

GEOMETRY INFORMATION INPUT:

PROPOSED TOP OF ROADWAY ELEV:				ft		m
PROPOSED TOP OF BACKWALL ELEV:			5971.45	ft	1820.56	m
PROPOSED BRIDGE SEAT ELEV:	H_Backwall = 2.17 ft		5969.28	ft	1819.90	m
PROPOSED TOP OF FOOTING ELEV:	H_Footing = 4.92 ft		5948.28	ft	1813.50	m
PROPOSED BOT. OF FOOTING ELEV:			5943.36	ft	1812.00	m
ELEVATION OF HIGH WATER:		FOR NO WATER = 0.00	5965.22	ft	1818.20	m
PROPOSED BRIDGE SEAT WIDTH:			3.28	ft	1.00	m
PROPOSED BACKWALL WIDTH:			0.00	ft	0.00	m
ABUTMENT/PIER/WALL DESIGN LENGTH:	1.00	Actual Length:	37.56	ft	11.45	m
FOOTING LENGTH		Actual Length:	42.48	ft	12.95	m

DW CALCULATION INPUT:

WEARING SURFACE DEPTH:	1.97 IN	x 1. Layers	0.16	ft <-- ---	0.050	m
ROADWAY WIDTH:			26.24	ft	8.000	m
BRIDGE SPAN:	Total Length = 79.84		39.92	ft	12.170	m
NUMBER OF GIRDERS:			1			

Fill-in for Abutment / Pier Design

MATERIAL PROPERTIES:

CUBIC WEIGHT CONCRETE:	150.00	pcf
COMP. STRENGTH OF CONC. = F'c:	4.00	ksi
MAXIMUM SIZE OF COARSE AGGREGATE	1.50	in
TENSILE STRENGTH OF REBAR = Fy:	60.00	ksi
CUBIC WEIGHT OF HOT MIX ASPHALT (HMA):	165.00	pcf

GEOTECHNICAL INFORMATION:

BEARING RESISTANCE (CAPACITY):	8.00	ksf	
NOMINAL BEARING RESISTANCE, qn :	26.00	ksf	<-- Per Geotech Report
WEIGHT OF SOIL BACKFILL:	130.00	Lbs/CF	<-- Assumed
WALL ON ROCK?	N	(Y OR N)	
WALL ON PILES?	N	(Y OR N)	
GRAVITY WALL?	N	(Y OR N)	
BETA: SLOPE OF BACKFILL:	0.00	DEG	<-- Assumed
THETA: BATTER ANGLE BACKWALL:	90.00	DEG	AASHTO Table 3.11.5.3-1
PHI: FRICTION ANGLE OF BACKFILL:	33.00	DEG	<-- Assumed
DELTA: ANGLE BACKWALL FRICTION:	22.00	DEG	<-- Assumed δ=2/3 ($\varnothing$)

CANTILEVER ABUTMENT DESIGN	N
GRAVITY ABUTMENT DESIGN	N
CANTILEVER WALL DESIGN	N
GRAVITY WALL DESIGN	N
PIER DESIGN	Y

PIER DESIGN -INPUT

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	ALH
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Pier	**Date:**	June 25, 2014

General Loading Parameters

Input Section : 2.0

LIVE LOAD INFORMATION:

APPROACH SLAB:	N	(Y OR N)
ROADWAY WITHIN H/2 OF TOP OF WALL:	Y	(Y OR N)
Live Load Surcharge to be Considered?:	N	
SURCHARGE HEIGHT:	0.00	ft REF: Table 3.11.6.4-1
Construction Surcharge, q:	250.00	psf REF: C3.4.2.1

SEISMIC LOAD INFORMATION:

WALL RESTRAINED HORZ. MOVMT.(Y/N):	N	(Y OR N)
SEISMIC ACCELERATION COEFF. A:	0.290	REF: FIG.3.10.2.1-2, AASHTO
SEISMIC CATEGORY:	D	**<---Assumed based on Location & AASHTO Seimic Design Guide**

RAILING CLASS: S3-TL4 (CT) (PER MASSDOT LRFD BRIDGE MANUAL PART 1) 3.3.2.2 **<--- N/A**

Horizontal Railing Design Load	0.00	kips
Horizontal Railing Impact Length	0.00	ft
Wall Height+Rail Height	0.00	ft
Distributed Horizontal Railing Design Load @ top of wall	0.00	klf
Distributed Horizontal Railing Design Load @ bottom of wall	0.00	klf/wall height
Railing Dead Load	0.00	
Additional Moment From Railing Impact	0.00	<--- **Note:** The added moment from top of railing to bottom of railing is distributed along bottom of footing*

STREAM PRESSURE

Pmax	0.00	psf
Consider Stream Flow:	N	<--- Do not include stream pressure for the wall.

SURCHARGE HEIGHT (Per ASSHTO 3.11.6.4 Live Load Surchage)

ABUTMENTS (N/A for PIERS) ----> Table 3.11.6.4-1

Table 3.11.6.4-1 - Equivalent Height of Soil for Vehicular Loading on Abutments Perpindicular to Traffic

Abutment Height (ft)	h_{eq} (ft)
5	4
10	3
>20	2

Surcharge Height = 0.00 ft

RETAINING WALLS ---> Table 3.11.6.4-2

See Table 3.11.6.4-2 for Equivalent Height of Soil for Vechicular Loading on Retaining Walls Parallel to Traffic.

Retaining Wall Height (ft)	heq (ft) Distance from wall backface to edge of traffic.	
	0.0 ft	≥ 1.0 ft
5	5	2
10	3.5	2
>20	2	2

Distance from wall backface to edge of traffic = 0.0 ft

Surcharge Height = 0.00 ft

Note: See 3.11.6.5 for Possible Reduction of Surcharge

PIER DESIGN -INPUT

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	ALH
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Pier	**Date:**	June 25, 2014

Superstructure Loading Parameters

Input Section : 3.0

ADDITIONAL LOADS ON STRUCTURE

(load is per linear foot of structure (Abutment/ Pier/ Wall) NOT the Footing, arm from front edge of bridge seat)

LOADS		LOAD (klf)	ARM (feet)			
(DC+DW), SUPERSTRUCT. DEAD LOAD:	DL	15.44	1.64	Distance from front face of the abutment/Pier/Wall to CL of bearing	Include =	Y
DC (Structural Components & nonstructural attachments)	DC	14.58	1.64	Distance from front face of the abutment/Pier/Wall to CL of bearing	Include =	Y
DW (Wearing Surface & Utilities)	DW	0.87	1.64	Distance from front face of the abutment/Pier/Wall to CL of bearing	Include =	Y
(LL+IM+PL), LIVE LOAD, IMPACT AND PED LL:	LL+IM+PL	9.81	1.64	Distance from front face of the abutment/Pier/Wall to CL of bearing	Include =	Y
WS, WIND LOAD ON STRUCTURE:	WS	0.00	0.00	Distance above the bridge seat where the longitudinal force is applied.	Include =	Y
WL, WIND LOAD ON LIVE LOAD:	WL	0.00	0.00	Distance above the bridge seat where the longitudinal force is applied.	Include =	Y
BR, BREAKING LOAD :	BR	0.96	0.00	Distance above the bridge seat where the longitudinal force is applied.	Include =	Y
TU, THERMAL FORCE:	TU	0.00	0.00	Distance above the bridge seat where the longitudinal force is applied.	Include =	Y
EQ, SEISMIC LOAD ON SUPERSTRUCTURE:	EQ	15.88	0.00	Distance above the bridge seat where the longitudinal force is applied.	Include =	Y
CT, VEHICLE COLLISION LOAD	CT	0.00	0.00	Distance above top pf wall equal to the height of rail	Include =	Y

Note: Per AASHTO 11.5.1, abutments and retaining walls should be designed for EH, WA, LS, DS, DC, TU, EQ. Therefore, including wind and breaking forces is conservative. Say OK

PIER DESIGN -INPUT

General Information

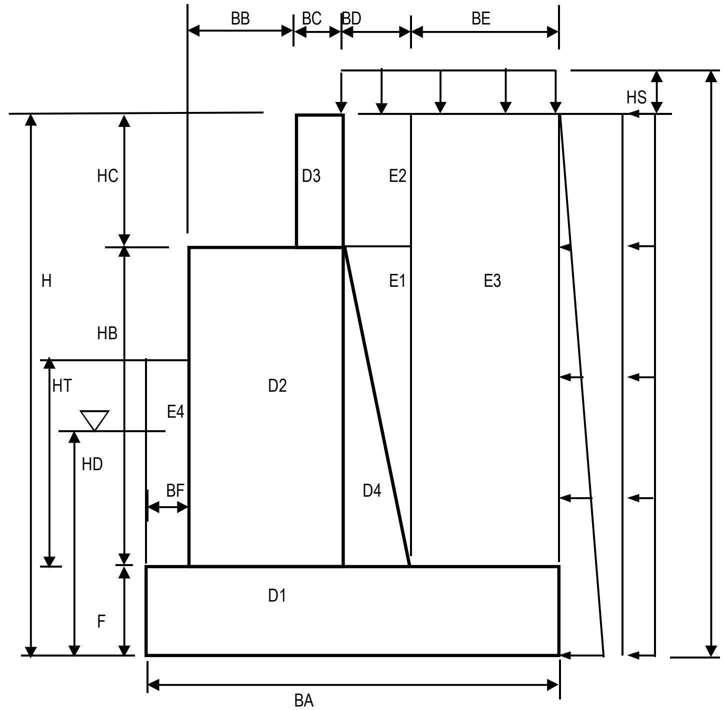
Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	ALH
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Pier	**Date:**	June 25, 2014

Abutment Geometry

Input Section : 4.0

CALCULATION OF WALL AND BACKFILL GEOMETRY:

		Prelim Size	User Adjust	Final Size (ft)	Approx Size (mm)
HEIGHT OF ABUTMENT / WALL, H:	H =	28.090	0.00	28.09	8500
HEIGHT OF FOOTING, F:	F =	4.920	0.00	4.92	1500
HEIGHT OF STEM, HB:	HB =	21.000	0.00	21.00	6300
HEIGHT OF BACKWALL, HC:	HC =	2.165	0.00	2.17	700
HEIGHT OF HIGH WATER, HD:	HD =	21.860	0.00	21.86	6600
HEIGHT OF SURCHARGE, HS:	HS =	0.000	0.00	0.00	0
WIDTH OF FOOTING, BA:	BA =	19.680	0.00	19.68	5910
WIDTH OF BRIDGE SEAT, BB:	BB =	3.281	0.00	3.28	990
WIDTH OF BACKWALL, BC:	BC =	0.000	0.00	0.00	0
WIDTH OF BATTER OF STEM, BD:	BD =	0.000	0.00	0.00	0
WIDTH OF FOOTING HEEL, BE:	BE =	8.200	0.00	8.20	2460
WIDTH OF FOOTING TOE, BF:	BF =	8.199	0.00	8.20	2460
HEIGHT OF SOIL OVER TOE, HT:	HT =	10.860	0.00	10.86	3260
HEIGHT OF SOIL OVER HEEL, HH:	HH =	10.860	0.00	10.86	3300
HEIGHT OF SOIL AT BACKFACE FACE (HEEL), HS1	Hss1 =	15.78	0.00	15.78	4800
HEIGHT OF SOIL AT FRONT FACE FACE (TOE), HS2	Hss2 =	15.78	0.00	15.78	4800



OVERALL QUANTITIES:

WEIGHT OF CONCRETE WALL/L.F.:	24.859	Kips per l.f.
CONCRETE QUANTITY / L.F.:	6.138	C.Y. per l.f.

SUMMARY OF QUANTITIES:

STEEL / L.F. =	944.144	LBS/L.F.
CONC. / L.F =	6.138	C.Y./L.F.

Geometry Check: Check Width: ok

Check Height: ok

PIER DESIGN - PRIMARY LOADS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	ALH
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Pier	**Date:**	June 25, 2014

References:
- AASHTO LRFD Bridge Design Specifications, 6th Edition, 2012
- ACI 318-08 Building Code Requirements for Structural Concrete, 2005
- 2009 MassDOT LRFD Bridge Manual, including draft Novemeber 2012 provisions
- AASHTO Guide Specifications for LRFD Seismic Bridge Design 2011

Notes:
This template assumes that the soils strata behind the abutment is uniform (only 1 strata is considered).
BRIDGE Design **Khost Bridge Notes**

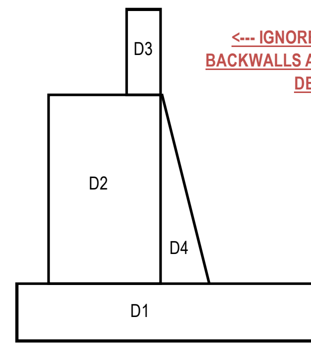
Calculate Dead Loads

Primary Loads Section : 1.0

Superstructure Loads: Vertical: / Horizontal:

AREA #		Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	Superstructure	14.58	9.84	143.44			
DW	Superstructure	0.87	9.84	8.51			

* See the load column under "Additional Loads on Structure" in the "General Loading Parameters" section for the above forces.



<--- IGNORE SECTION D3, BACKWALLS ARE N/A FOR PIER DESIGN

Substructure Loads: Vertical: / Horizontal:

AREA #		Volume (CF)	γ_{conc} (pcf)	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	D1	96.83	150.00	14.52	9.84	142.91			
	D2	68.90	150.00	10.34	9.84	101.69			
	D3	0.00	150.00	0.00	11.48	0.00			
	D4	0.00	150.00	0.00	11.48	0.00			
	Subtotal Concrete			**24.86**		**244.61**			

<-- N/A FOR PIER DESIGN
<-- N/A, NO BATTER FOR THIS DESIGN

Total Dead Load: Vertical: / Horizontal:

AREA #	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
TOTAL DC (Super + Sub)	**39.44**		**388.05**			
TOTAL DW (Super)	**0.87**		**8.51**			
TOTAL DC (Substr. Only - Construction)	**24.86**		**244.61**			

PIER DESIGN - PRIMARY LOADS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	ALH
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Pier	**Date:**	June 25, 2014

Calculate Earth Loads

Primary Loads Section : 2.0

Compute Horizontal Earth Pressure, EH:

Coulomb's Active Earth Pressure: (per MHD 3.1.5 and AASHTO 3.11.5.3)

PHI, ϕf =	33.00	Degrees, Rad =	0.58
DELTA, δ =	22.00	Degrees, Rad =	0.38
BETA, β =	0.00	Degrees, Rad =	0.00
THETA, θ =	90.00	Degrees, Rad =	1.57
Γ (per AASHTO Eq. 3.11.5.3-2)=	2.87		
Ka (per AASHTO Eq. 3.11.5.3-1)=	**0.264**		

Earth Pressure Coefficient to be Used for Design per MassDOT

All Walls on Rock	ko	0.455	
All Walls on Piles	ko	0.455	
Cantilever Walls < than 16' in Height	0.5*(Ko + Ka)	0.360	
Cantilever Walls > than 16' in Height	Ka	0.264	<-- USE
Gravity wall supported on Spread Footing	Ka	0.264	

At-Rest Earth Pressure Coeff:

Ko = **0.455**

Earth Pressure Coefficient to be Used for Design: 'Active pressure coefficients shall be estimated using Coulomb Theory.

WALL ON LEDGE:	N	(Y OR N)
WALL ON PILES:	N	(Y OR N)
Wall Height:	28.09	ft
Earth pressure Type:	Ka	
Ke =	**0.264**	<=== **Does not govern.**

Earth Pressure Coefficients to be Used for Design per Geotechnical Report:

Ko =	0.49	
Ka =	0.32	
Ke (geotech) =	**0.320**	<===== **Governs.**

Compute Lateral Earth Pressure:

Application of lateral earth pressure shall be per AASHTO Figure C3.11.5.3-1. This shows a different application for Gravity and Cantilever (semi-gravity) walls.
Note that the reduction in lateral earth pressures due to the water table is not included in this section. It is included in the WA (Bouyancy) section of this design.

Cantilever (semi-gravity) Walls:

THIS SECTION IS FOR CANTILEVER OR SEMI-GRAVITY WALLS ONLY

Load inclination from horizontal, min = $\phi/3$ =	11.00	degrees
Load inclination from horizontal, max = $\phi*2/3$ =	22.00	degrees
GAMMA =	130.00	pcf
H = Soil Height at Back face, Hss1-1'	**14.78**	Feet
Lateral Earth Load, Pa = 1/2*Ke*γ*H^2 =	4.54	kips
Arm for Horiz Load above BOF = H/3 =	4.93	ft
Arm for Vert Load from Toe = F =	19.68	ft

Consider minimum inclination for Sliding, Overturning and Bearing Pressure:

Vertical Component, Pav = Pa*sin(ϕ/3) =	0.87	klf
Horizontal Component, Pah = Pa*cos(ϕ/3) =	4.46	klf

Consider maximum inclination for Footing Heel Reinforcement:

Vertical Component, Pav = Pa*sin(ϕ*2/3) =	1.70	klf
Horizontal Component, Pah = Pa*cos(ϕ*2/3) =	4.21	klf

Gravity Walls:

N/A ---> THIS SECTION IS FOR GRAVITY WALLS ONLY

Load inclination from horizontal = $\delta + (90-\theta)$ =	22.00	degrees
GAMMA =	130.00	pcf
H =	14.78	Feet
Lateral Earth Load, Pa = 1/2*Ke*γ*H^2 =	4.54	kips
Arm for Horiz Load above BOF = H/3 =	4.93	ft
Arm for Vert Load from Toe=(BF+BB+BC+BD*2/3) =	11.48	ft

Consider for Sliding, Overturning, Bearing Pressure and Footing Reinforcement:

Vertical Component, Pav = Pa*sin($\delta+(90-\theta)$) =	1.70	klf
Horizontal Component, Pah = Pa*cos($\delta+(90-\theta)$) =	4.21	klf
Is the wall a Gravity Wall?	N	

PIER DESIGN - PRIMARY LOADS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	ALH
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Pier	**Date:**	June 25, 2014

Calculate Earth Loads Continued..

Primary Loads Section : 2.1

Include Passive Earth Pressure	Y
Pp Factor	1

ϕ = Soil Friction Angle	33.00	degrees
δ = Wall Interface Friction	22.00	degrees = 2/3 * ϕ --> 11.6.5.5
Kp = Passive Earth Pressure Coefficient	3.13	Fig A11.4-2
γ = Unit Weight of Soil	130.00	pcf
H = Hss2= Height of Soil at Front Face - 1'	14.78	ft

Lateral EQ Load, Pp = 1/2*γ*Kp*H^2=	44.44	klf > Pah -------> Use Pp = Pah --------->	P_p = 4.46 klf
Arm for Horiz Load above BOF = H/3 =	4.93	ft (AASHTO pg 11-112)	

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	ALH
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Pier	**Date:**	June 25, 2014

Calculate Earth Loads Continued..

Primary Loads Section : 2.2

Forces From Earth Retention, EH:

Active Pressure Force Inclination	AREA #	Vertical: Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horizontal: Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
Condition of Minimum Inclination of Active Earth Pressure. (This condition to be used for designs other than heel reinf.)	Forces From Active Earth Pressure	0.87	19.68	17.06	4.46	4.93	21.97
	Forces From Passive Earth Pressure			0.00	4.46	4.93	21.97
	EH: Due to Cantilevered Wingwalls (QTY 2)	0.00	0.00	0.00	0.00	0.00	0.00
	When Passive Pressure Considered.	0.87	19.68	17.06	0.00	0.00	0.00
	When Passive Pressure Not Considered.	0.87	19.68	17.06	4.46	4.93	21.97
	Consider Passive Pressure To Counteract The Active Pressure From The Retained Earth						
	Controlling Earth Pressures	0.87	19.68	17.06	0.00	0.00	0.00
Condition of Max Inclination of Active Earth Pressure. (This condition used for heel reinf. Design only)	**Earth Pressures For Heel Reinforcement Design**	1.70	19.68	33.50	4.21	4.93	20.76

<== See attached Calculations: Earth Load on Abutment due to wingwalls

<=== Note, Based on AASHTO Figure C11.5.6-1, both the vertical and horizontal compon

Vertical Earth Pressure, EV:

	AREA #	Volume (CF)	γ_{SOIL} (plf)	Vertical: Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horizontal: Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
EV	E1	0.00	130.00	0.00	11.48	0.00			
	E2	0.00	130.00	0.00	11.48	0.00			
	E3	189.99	130.00	24.70	15.58	384.81			
	E4	89.04	130.00	11.58	4.10	47.45			
TOTAL EV				**36.27**		**432.27**			

<-- N/A Batter = 0 (E1)

<-- N/A Batter = 0 (E2)

Note, **per AASHTO 11.6.1.2**, the weight of the soil over the battered portion of the stem or over the base of a footing may be considered as part of the effective weight of the abutment. This is consistant with design.

Earth Surcharge, ES: (This applies for construction case only)

q =	250.00	psf
Uniform Load on Wall, p=Ke*q =	0.080	ksf
Wall Height, H =	28.09	Feet
Heel Length, BE =	8.20	Feet
Footing Width, BA =	19.68	Feet
Wall Length Considered =	1.00	ft

	AREA #	Vertical: Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horizontal: Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
ES	$P_{con}(h)$ = p*H*Length =				2.25	14.05	31.56
	$P_{con}(v)$ = q*BE*Length =	2.05	15.58	31.94			
TOTAL ES		**2.05**		**31.94**	**2.25**		**31.56**

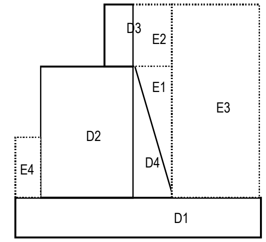
PIER DESIGN - PRIMARY LOADS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	ALH
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Pier	**Date:**	June 25, 2014

Calculate Live Loads

Primary Loads Section : 3.0



<--- IGNORE SECTION D3, BACKWALLS ARE N/A FOR PIER DESIGN

Superstructure Loads: Vertical: Horizontal:

AREA #		Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
LL+IM+PL	Superstructure	9.81	9.84	96.50			
BR	Superstructure				0.959	25.93	24.85

* See the load column under "Additional Loads on Structure" in the "General Loading Parameters" section for the above forces.

Live Load Surcharge Loads: LS

Per AASHTO 3.11.6.4, a live load surcharge shall be applied where vehicular load is expected to act on the surface of the backfill within a distance equal to one-half the wall height behind the back face of the wall.
If the surcharge is for highway, the intensity of the load shall be consistent with provisions of Article 3.6.1.2. See Tables 3.11.6.4-1 and 3.11.6.4-2 for equivalent heights.

Compute Horizontal Live Load Surcharge: (To be used for bearing pressure and sliding load cases):

Ke =	0.264	
Unit Weight of Soil, γ =	130.000	pcf
Surcharge Height, heq =	0.00	Feet
LS(h) =(Ke)(γ)(heq)*H =	0.00	kips
Moment arm = H/2 =	14.05	kips

Compute Vertical Live Load Surcharge: (To be used for bearing pressure cases only):

LS(v) =(γ)(heq)(BD+BE) =	0.00	kips
Moment arm = Ba-(BD+BE)/2 =	15.58	kips

Compute Vertical Live Load Surcharge: (To be used for heel reinf cases only):

LS(v) =(γ)(heq)(BE) =	0.00	kips
Moment arm (to back of batter) = BE/2 =	4.10	kips

Live Load Surcharge, LS: Summary Vertical: Horizontal:

AREA #		Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
LS	LS(v)	0.00	15.58	0.00			
	LS(h)				0.00	14.05	0.00

Total Live Load Load: Vertical: Horizontal:

AREA #	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
TOTAL LL+IM+PED+BR+LS	9.81		96.50	0.96		24.85
TOTAL LL+IM+PED+BR+LS (Sliding Only)	9.81		96.50	0.96		24.85
TOTAL LS (Heel Reinf Only)	0.00	4.10	0.00			

PIER DESIGN - PRIMARY LOADS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	ALH
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Pier	**Date:**	June 25, 2014

Calculate Water load (Buoyancy Forces)

Primary Loads Section : 4.0

HEIGHT OF STEM AT HIGH WATER:	16.94	
HEIGHT OF FOOTING AT HIGH WATER:	4.92	
WIDTH OF FOOTING, BA	19.68	
SOIL WEIGHT - WATER WEIGHT	67.60	pcf
UPWARD BOUYANT FORCE	-62.40	pcf

Horizontal Force = B(h) = (γ-(γ-62.4))*Ka)H^2/2, acts at HD/3:

INCLUDE HORIZONTAL FORCE? N

<-- Note: The Horizontal load is Not Applicable since the hydrostatic force is equal and opposite on both sides.

Bouyant Load, WA:

AREA #		VOLUME (CF)	GAMMA (#/CF)	Vertical: Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horizontal: Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
WA	B1 (Ftg)	96.83	-62.40	-6.04	9.84	-59.45			
	B2 (Stem)	55.58	-62.40	-3.47	9.84	-34.13			
	B3 (Soil over Ftg)	277.80	-62.40	-17.33	15.58	-270.07			
	STATIC						4.77	7.29	34.76
	SEISMIC						10.90	7.29	79.42
TOTAL WA (BL) (Static)				**-26.84**		**-363.65**	**0.00**		**0.00**
TOTAL WA (BL) (Seismic)				**-26.84**		**-363.65**	**0.00**		**0.00**

Calculate Stream Flow Pressure

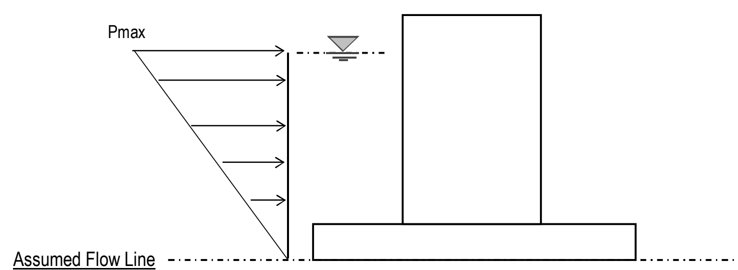
Primary Loads Section : 4.1

Note: The flow line is conservatively assumed to act at the bottom of the footing

Pmax:	0.0000	ksf
APPLIED:	N	

Force = 0.5 * Pmax * HD
Arm = HD * (2/3)

LOAD	HORIZONTAL: **FORCE (Kips)**	**ARM (Feet)**	**MOM (Ft x K)**
WA (SF)	0.00	14.57	0.00



Calculate Water Load & Stream Flow Load WA

Primary Loads Section : 4.2

Water Load (Bouyancy) & Stream Flow, WA:

AREA #	VOLUME (CF)	GAMMA (#/CF)	Vertical: Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horizontal: Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
TOTAL WA (Static)			-26.84		-363.65	0.00		0.00
TOTAL WA (Seismic)			-26.84		-363.65	0.00		0.00

PIER DESIGN - PRIMARY LOADS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	ALH
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Pier	**Date:**	June 25, 2014

Calculate Wind Loads

Primary Loads Section : 5.0

Superstructure Loads: Vertical: Horizontal:

AREA #		Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
WS	**Superstructure**				**0.00**	**25.93**	**0.00**
WL	**Superstructure**				**0.00**	**25.93**	**0.00**

* See the load column under "Additional Loads on Structure" in the "General Loading Parameters" section for the above forces.

Calculate Temperature Loads

Primary Loads Section : 6.0

Superstructure Loads: Vertical: Horizontal:

AREA #		Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
TU	**Superstructure**				**0.00**	**25.93**	**0.00**

* See the load column under "Additional Loads on Structure" in the "General Loading Parameters" section for the above forces.

PIER DESIGN - PRIMARY LOADS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	ALH
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Pier	**Date:**	June 25, 2014

Calculate Seismic Forces

Primary Loads Section : 7.0

Superstructure Loads: Vertical: Horizontal:

AREA #		Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
EQ	**Superstructure**				**15.881**	**25.93**	**411.70**

* See the load column under "Additional Loads on Structure" in the "General Loading Parameters" section for the above forces.

Substructure Loads:

(Ref: AASHTO 4th Ed., A11.1.1.1 for Mononobe-Okabe Analysis.)

GAMMA = unit weight of soil =	130.00	Lbs/CF		
H = height of soil face =	28.09	Feet		
PHI = angle of internal friction of soil =	33.00	Degrees =	0.58	Radians
DELTA = angle of friction between soil & abut =	22.00	Degrees =	0.38	Radians
i = backfill slope angle =	0.00	Degrees =	0.00	Radians
BETA = slope of wall to the vertical	0.00	Degrees =	0.00	Radians

A =	0.29			
kh = horizontal acceleration coefficient	0.435	Consider Cohesion?	N	------> kh = a * 0.5, Wall is NOT Restrained from Horizontal Movement
kv = vertical acceleration coefficient	0.000			
THETA = arc tan (kh/(1-Kv) =	23.51	Degrees =	0.41	Radians
Kae (per AASHTO Eq. A11.1.1.1-2) =	**0.731**	<===== **Governs.**		

Earth Pressure Coefficients to be Used for Design per Geotechnical Report:

Kae (geotech) = 0.000 <=== **Does not govern.**

N/A
NOT GIVEN IN GEOTECH REPORT

Load inclination from horizontal = δ =	22.00	degrees
Lateral EQ Load, Eae = 1/2*γ*Kae*H^2*(1-kv) =	37.49	klf
Arm for Horiz Load above BOF = H/3 =	9.36	ft (AASHTO pg 11-112)
Arm for Vert Load from Toe = BA =	19.68	ft

Consider for Sliding, Overturning, Bearing Pressure and Footing Reinforcement:

Vertical Component, Eav = Eae*sin(δ) =	14.05	klf	Include EQ In Design =	Y
Horizontal Component, Eah = Eae*cos(δ) =	34.76	klf	EQ Factor =	1

PIER DESIGN - PRIMARY LOADS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	ALH
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Pier	**Date:**	June 25, 2014

Calculate Seismic Forces

Primary Loads Section : 7.1

Include Seismic Passive Earth Pressure	Y	
Epe Factor	1	
kh = horizontal acceleration coefficient	0.435	
∅ = Soil Friction Angle	33.00	degrees
δ = Wall Interface Friction	22.00	degrees = 2/3 * ∅ --> 11.6.5.5
Kpe = Seismic Passive Earth Pressure Coefficient	3.13	Fig A11.4-2
γ = Unit Weight of Soil	130.00	pcf
Hff = Height of Soil at Front Face -1'	14.78	ft
Lateral EQ Load, Epe = 1/2*γ*Kpe*H^2=	44.44	klf ---> Equation A11.4-4
Horizontal Component, Eah (calculated earlier) =	34.76	klf > Kpe Calculated Above
=====> Use Epe =	34.76	klf
Arm for Horiz Load above BOF = Hff/3 =	4.93	ft (AASHTO pg 11-112)

SECTION 11: WALLS, ABUTMENTS, AND PIERS 11-117

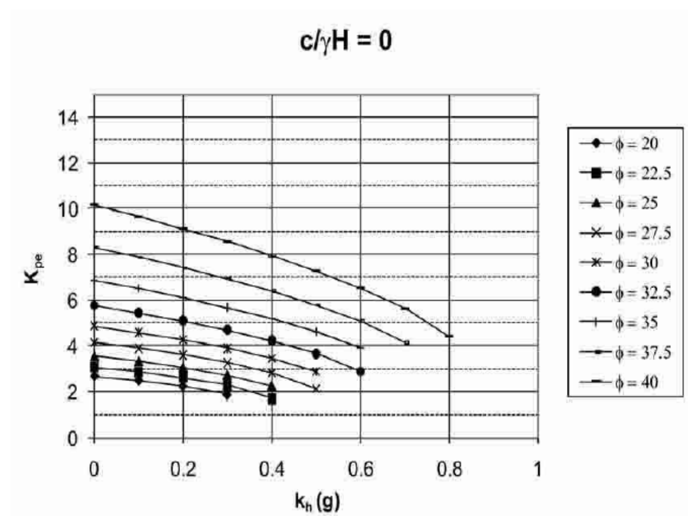


Figure A11.4-2—Seismic Passive Earth Pressure Coefficient Based on Log Spiral Procedure for $c/\gamma H$ = 0 and 0.05 (c = soil cohesion, γ = soil unit weight, and H = height or depth of wall over which the passive resistance acts)

Note: $k_h = A_s = k_{ho}$ for wall heights greater than 20 ft.

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	ALH
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Pier	**Date:**	June 25, 2014

Calculate Seismic Forces Continued..

Primary Loads Section : 7.2

WALL INERTIA EFFECTS

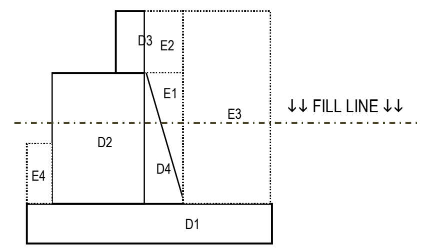
Per **AASHTO DIV 1A 6.4.3**, seismic design should take into account forces arising from seismically inducd lateral earth pressures (as computed above), additional forces arising from wall inertia and the transfer of seismic forces from the bridge deck through bearing supports which do not slide freely.

The following table computes the inertia forces due to the weight of the concrete and backfill.

kh = 0.435

FOR PIERS: Include DL above Fill Only

AREA #		DL (Kips)	DL*kh (Kips)	ARM (Feet)	MOM (Ft x K)	% of DL to be included
DL Wall	D1	14.52	6.32	2.46	15.54	100%
	D2	5.34	2.32	15.42	35.85	52%
	D3	0.00	0.00	27.00	0.00	100%
	D4	0.00	0.00	11.92	0.00	100% n/a
	Subtotal	**19.87**	**8.64**	**5.95**	**51.39**	
DL Backfill	E1	0.00	0.00	18.92	0.00	100%
	E2	0.00	0.00	27.00	0.00	100%
	E3	24.70	10.74	16.50	177.31	100%
	E4	11.58	5.04	10.35	52.12	100%
	Subtotal	**36.27**	**15.78**	**14.54**	**229.42**	
TOTAL		**56.14**	**24.42**	**11.50**	**280.81**	



Total Seismic Loads, EQ:

AREA #		Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
EQ	EQ Superstructure =				15.881	25.93	411.702
	Eae(v)	14.05	19.68	276.41			
	Eae(h)				34.76	9.36	325.50
	Epe(v)		19.68	0.00			
	Epe				-34.76	4.93	-171.27
	Fwi(h)				24.42	11.50	280.81
TOTAL EQ		**14.05**		**276.41**	**40.30**		**846.75**

% Eae(h) to be included:

100% **FOR PIERS: M-O ANALYSIS IS FOR RETAINED SOILS --> N/A FOR PIERS**

PIER DESIGN - PRIMARY LOADS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	ALH
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Pier	**Date:**	June 25, 2014

Calculate Vehicle Collision Loads

Primary Loads Section : 8.2

Superstructure Loads: Vertical: Horizontal:

AREA #		Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
CT (Stem Design)	**Superstructure**				**0.00**	**0.00**	**0.00**
CT	**Superstructure**				**0.00**	**0.00**	**0.00**

* See the load column under "Additional Loads on Structure" in the "General Loading Parameters" section for the above forces.

Summary of Primary Loads

Primary Loads Section : 9.2

		Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
TOTAL DC (Super + Sub)		39.44		388.05			
TOTAL DW (Super)		0.87		8.51			
TOTAL DC (Substr. Only - Construction)		24.86		244.61			
Controlling Earth Pressures		0.87	19.68	17.06	0.00	0.00	0.00
Earth Pressures For Heel Reinforcement Design		1.70	19.68	33.50	4.21	4.93	20.76
TOTAL EV		36.27		432.27			
TOTAL ES		2.05		31.94	2.25		31.56
TOTAL LL+IM+PED+BR+LS		9.81	0.00	96.50	0.96	0.00	24.85
TOTAL LL+IM+PED+BR+LS (Sliding Only)		9.81	0.00	96.50	0.96	0.00	24.85
TOTAL LS (Heel Reinf Only)		0.00	4.10	0.00	0.00	0.00	0.00
TOTAL WA (Static)		-26.84		-363.65	0.00		0.00
TOTAL WA (Seismic)		-26.84		-363.65	0.00		0.00
WS	Superstructure				0.00	25.93	0.00
WL	Superstructure				0.00	25.93	0.00
TU	Superstructure				0.00	25.93	0.00
TOTAL EQ		14.05		276.41	40.30		846.75
CT (Stem Design)		0.00	0.00	0.00	0.00	0.00	0.00
CT		0.00	0.00	0.00	0.00	0.00	0.00

PIER DESIGN - LOAD COMBINATIONS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	ALH
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Pier	**Date:**	June 25, 2014

References:
- AASHTO LRFD Bridge Design Specifications, 6th Edition, 2012
- ACI 318-08 Building Code Requirements for Structural Concrete, 2005
- 2009 MassDOT LRFD Bridge Manual, including draft Novemeber 2012 provisions
- AASHTO Guide Specifications for LRFD Seismic Bridge Design 2011

Notes:
This template assumes that the soils strata behind the abutment is uniform (only 1 strata is considered).

Khost Bridge Notes

Summary of Primary Loads

Load Combinations : 1.0

INCLUDE SEISMIC = Y

Load		Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)	Notes	LRFD Load Combination Load Case
Dead Load	$DC_{SUB+SUPER}$	39.44	0.00	388.05	0.00	0.00	0.00	Super + Sub	
	DW	0.87	0.00	8.51	0.00	0.00	0.00	Super Only	
	DC_{SUB}	24.86	0.00	244.61	0.00	0.00	0.00	Sub Only - Construction	LC1 only
Earth Load	EH	0.87	19.68	17.06	0.00	0.00	0.00	All cases except Heel	Used in all load cases
	EH	1.70	19.68	33.50	4.21	4.93	20.76	For Heel Reinforcement	Not used in any load case
	EV	36.27	0.00	432.27	0.00	0.00	0.00		
Earth Load Surcharge	ES	2.05	0.00	31.94	2.25	0.00	31.56		
Live Load Surcharge	LS(v)	0.00	15.58	0.00	0.00	0.00	0.00		
	LS(h)	0.00	0.00	0.00	0.00	14.05	0.00		
Live Load	LL+IM+PED+BR+LS	9.81	0.00	96.50	0.96	0.00	24.85		
	LL+IM+PED+BR+LS	9.81	0.00	96.50	0.96	0.00	24.85	No LS for Sliding LC	LC4, LC8 & LC10
	LS	0.00	4.10	0.00	0.00	0.00	0.00		
Bouyant Load & Stream Force	WA	-26.84	0.00	-363.65	0.00	0.00	0.00	Static	
	WA	-26.84	0.00	-363.65	0.00	0.00	0.00	Seismic	LC9 &LC10
Wind Load	WS	0.00	0.00	0.00	0.00	25.93	0.00		
	WL	0.00	0.00	0.00	0.00	25.93	0.00		
Temperature Load	TU	0.00	0.00	0.00	0.00	25.93	0.00		
Seismic Load	EQ	14.05	0.00	276.41	40.30	0.00	846.75		
Vehicle Collision Load	CT	0.00	0.00	0.00	0.00	0.00	0.00	Stem Wall	LC11 & LC12
	CT	0.00	0.00	0.00	0.00	0.00	0.00	Stability	

PIER DESIGN - LOAD COMBINATIONS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	ALH
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Pier	**Date:**	June 25, 2014

Limit States and Load Factors

Load Combinations : 2.0

Service Limit State

Per AASHTO 10.5.2, foundation design at the service limit state shall include settlements, horizontal movements, overall stability (of earth slopes) and scour at the design flood.
* These items are part of the geotechnical scope and are therefore NOT included in this design.

Strength Limit States

Per AASHTO 10.5.3, foundation design at the strength limit strength shall include structural resistance, scour, nominal bearing resistance, overturning or excessive loss of contact, sliding and constructability.
* These items, except scour, are addressed in this design.

Extreme Events Limit States

Per AASHTO 10.5.4, foundation shall be designed for extreme events such as a seismic event and vehicle collision.
* These items are addressed in this design.

Since these factors are 1.0, they have not yet been incorporated into the design template.

Computation of the Load Modification Factor, h_i:

		Extreme	Strength
h_D Ductility Factor, (AASHTO 1.3.3):		1.00	1.00
h_R Redundancy Factor, (AASHTO 1.3.4):		1.00	1.00
h_I Operational Importance Factor, (AASHTO 1.3.5):		1.00	1.00
h_i (for loads for which γ_i(max) is appropriate) (AASHTO Eq 1.3.2.1-2):	$h_i = h_D h_R h_I \geq 0.95$	1.00	1.00
h_i (for loads for which γ_i(min) is appropriate) (AASHTO Eq 1.3.2.1-3):	$h_i = 1 / h_D h_R h_I \leq 1.00$	1.00	1.00

h_D Ductility Factor (for all other limit states $h_D = 1.00$)

$h_D \geq$	1.05	for nonductile components and connections.
$h_D =$	1.00	for conventional designs and details complying with the specifications.
$h_D \geq$	0.95	for components and connections for which additional ductility-enhancing m

h_R Redundancy Factor (for all other limit states $h_R = 1.00$)

$h_R \geq$	1.05	for nonredundant members
$h_R =$	1.00	for conventional levels of redundancy
$h_R \geq$	0.95	for exceptinal levels of redundancy

h_I Operational Importance Factor

$h_I \geq$	1.05	for a bridge of operational importance
$h_I =$	1.00	for typical bridges
$h_I \geq$	0.95	for relatively less important bridges

Load Factors for Permanent Loads (per AASHTO Table 3.4.1-2), g_p:

	Maximum	Minimum	
DC (Dead Load, General):	1.25	0.90	
DW (Wearing Surface & Utilities):	1.50	0.65	
EH (Horiz Earth):	1.43	0.90	<-- An average of Active and At-rest Coefficients used based on MHD's earth pressure design guidelines.
ES (Horiz Earth):	1.50	0.75	
EV (Vertical Earth, Retaining Structure):	1.35	1.00	

Live Load Factor During a Seismic Event, g_{EQ}:

	Maximum	Minimum	
g_{EQ} (AASHTO C3.4.1):	0.50	0.00	<--- Seismic Included

PIER DESIGN - LOAD COMBINATIONS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	ALH
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Pier	**Date:**	June 25, 2014

LRFD Load Combinations & Notes

Load Combinations : 3.0

NOTES:

1. Load Combination Strength II does not need to be checked since it applies to special design vehicles.
2. Load Combination Strength III does not need to be checked during construction since WS is not a significant load.
3. Load Combination Strength IV does not need to be checked since it applies to bridges with very high dead load to live load ratios.
4. Load Combination Strength V does not need to be checked during construction since WS and WL are not significant loads.
5. Extreme Event load combinations do not need to be checked during construction.
6. Extreme Event II load combinations does not need to be checked for abutments.
7. Service limit state load combinations do not need to be checked for abutment stability / reinforcement.
8. Fatigue limit state load combinations do not need to be checked for abutment stability / reinforcement.
9. All remaining load cases shall be checked using load factors which would provide max effect for either bearing or sliding / eccentricity similar to AASHTO Figures C11.5.5-1 and C11.5.5.2.
10. Bouyancy has been included in sliding load combinations. A load factor of 0.0 has been used for bearing pressure load combinations since it is conservative to ignore sliding for these computations.

Strength	LC1	LC1 - STRENGTH I CONSTRUCTION (Before Bridge Construction): gp max*(DCsub)+gp max*(EH)+gp max*(EV)+γp max*(ES)
Strength	LC2	LC2 - STRENGTH I CONSTRUCTION (Before Bridge LL): gp max*(DC+DW)+gp max*(EH)+gp max*(EV)+γp max*(ES)
Bearing	LC3	LC3 - STRENGTH I BEARING: gp max*(DC+DW)+gp max*(EH)+gp max*(EV)+1.75*(LL+IM+PL+BR+LS)+1.0*(WA)+0.50*(TU)
Sliding	LC4	LC4 - STRENGTH I SLIDING: gp min*(DC+DW)+gp max*(EH)+gp min*(EV)+1.75*(LL+IM+PL+BR+LS)+1.0*(WA)+0.50*(TU)
Bearing	LC5	LC5 - STRENGTH III BEARING: gp max*(DC+DW)+gp max*(EH)+gp max*(EV)+1.0*(WA)+1.4*(WS)+0.50*(TU)
Sliding	LC6	LC6 - STRENGTH III SLIDING: gp min*(DC+DW)+gp max*(EH)+gp min*(EV)+1.0*(WA)+1.4*(WS)+0.50*(TU)
Bearing	LC7	LC7 - STRENGTH V BEARING: gp max*(DC+DW)+gp max*(EH)+gp max*(EV)+1.35*(LL+IM+PL+BR+LS)+1.0*(WA)+0.4*(WS)+1.0*(WL)+0.50*(TU)
Sliding	LC8	LC8 - STRENGTH V SLIDING: gp min*(DC+DW)+gp max*(EH)+gp min*(EV)+1.35*(LL+IM+PL+BR+LS)+1.0*(WA)+0.4*(WS)+1.0*(WL)+0.50*(TU)
Extreme Bearing	LC9	LC9 - EXTREME EVENT I BEARING: gp max*(DC+DW)+gp max*(EH)+gp max*(EV)+gEQ MAX*(LL+IM+PL+BR+LS)+1.0*(EQ)
Extreme Sliding	LC10	LC10 - EXTREME EVENT I SLIDING: gp min*(DC+DW)+gp max*(EH)+gp min*(EV)+gEQ MIN*(LL+IM+PL+BR+LS)+1.0*(WA)+1.0*(EQ)
Extreme Bearing	LC11	LC11 - EXTREME EVENT II BEARING: gp max*(DC+DW)+gp max*(EH)+gp max*(EV)+0.50*(LL+IM+PL+BR+LS)+1.0*(CT)
Extreme Sliding	LC12	LC12 - EXTREME EVENT II SLIDING: gp min*(DC+DW)+gp max*(EH)+gp min*(EV)+0.50*(LL+IM+PL+BR+LS)+1.0*(WA)+1.0*(CT)

PIER DESIGN - LOAD COMBINATIONS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	ALH
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Pier	**Date:**	June 25, 2014

LRFD Load Combinations

Load Combinations : 3.1

↓ N/A, Valid for Pile Design Only ↓

NA (for Bottom row of piles) From Pile Design = 0

Bottom Row to Edge of Toe = 0

↓ N/A, Valid for Pile Design Only ↓

Distance of Pile Group N.A. From Footing Toe (See Pile Design Spreadsheet): 0.00 ft

LC1 - STRENGTH I CONSTRUCTION (Before Bridge Construction): $g_{p\,max}$*(DCsub)+$g_{p\,max}$*(EH)+$g_{p\,max}$*(EV)+$\gamma_{p\,max}$*(ES)

LOAD	Load Factor	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC_{SUB}	1.25	31.07		305.76	0.00		0.00
EH	1.43	1.24		24.31	0.00		0.00
EV	1.35	48.97		583.56	0.00		0.00
ES	1.50	3.08		47.91	3.37		47.34
SUM		**84.35**		**961.54**	**3.37**		**47.34**

Distance of Vertical Force (V) From The Footing Toe	Offset of Pile Group N.A. From Original Location of V	Equivalent Moment Due to Offset of Pile Group N.A. From Original Location of V	**Mom. to Be Used On Pile Group = O.T. Mom. - Equivalent Mom.**	**Vertical Force to Be Used On Pile Group**	**Horizontal Force to Be Used On Pile Group**
11.40 ft	11.40 ft	961.5 k.ft	**-914.2 k.ft**	**84.4 kip**	**3.4 kip**

LC2 - STRENGTH I CONSTRUCTION (Before Bridge LL): $g_{p\,max}$*(DC+DW)+$g_{p\,max}$*(EH)+$g_{p\,max}$*(EV)+$\gamma_{p\,max}$*(ES)

LOAD	Load Factor	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	1.25	49.30		485.06	0.00		0.00
DW	1.5	1.30		12.77	0.00		0.00
EH	1.43	1.24		24.31	0.00		0.00
EV	1.35	48.97		583.56	0.00		0.00
ES	1.50	3.08		47.91	3.37		47.34
SUM		**103.88**		**1153.62**	**3.37**		**47.34**

↓ N/A, Valid for Pile Design Only ↓

11.11 ft	11.11 ft	1153.6 k.ft	**-1106.3 k.ft**	**103.9 kip**	**3.4 kip**

PIER DESIGN - LOAD COMBINATIONS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	ALH
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Pier	**Date:**	June 25, 2014

LRFD Load Combinations Cont.

Load Combinations : 3.2

LC3 - STRENGTH I BEARING: $g_{p\,max}$*(DC+DW)+$g_{p\,max}$*(EH)+$g_{p\,max}$*(EV)+1.75*(LL+IM+PL+BR+LS)+1.0*(WA)+0.50*(TU)

LOAD	Load Factor	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	1.25	49.30		485.06	0.00		0.00
DW	1.5	1.30		12.77	0.00		0.00
EH	1.43	1.24		24.31	0.00		0.00
EV	1.35	48.97		583.56	0.00		0.00
LL+IM+PL+BR+LS	1.75	17.16		168.87	1.68		43.49
WA	1.00	-26.84		-363.65	0.00		0.00
TU	0.50	0.00		0.000	0.0000		0.000
SUM		**91.12**		**910.93**	**1.68**		**43.49**

Load Factors Based on this particular LRFD Combination

↓ N/A, Valid for Pile Design Only ↓

10.00 ft	10.00 ft	910.9 k.ft	**-867.4 k.ft**	**91.1 kip**	**1.7 kip**

LC4 - STRENGTH I SLIDING: $g_{p\,min}$*(DC+DW)+$g_{p\,max}$*(EH)+$g_{p\,min}$*(EV)+1.75*(LL+IM+PL+BR+LS)+1.0*(WA)+0.50*(TU)

LOAD	Load Factor	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	0.9	35.49		349.24	0.00		0.00
DW	0.65	0.56		5.53	0.00		0.00
EH	1.43	1.24		24.31	0.00		0.00
EV	1.00	36.27		432.27	0.00		0.00
LL+IM+PL+BR+LS	1.75	17.16		168.87	1.68		43.49
WA (static)	1.00	-26.84		-363.65	0.00		0.00
TU	0.50	0.00		0.00	0.000		0.000
SUM		**63.88**		**616.58**	**1.68**		**43.49**

Load Factors Based on this particular LRFD Combination

↓ N/A, Valid for Pile Design Only ↓

9.65 ft	9.65 ft	616.6 k.ft	**-573.1 k.ft**	**63.9 kip**	**1.7 kip**

LC5 - STRENGTH III BEARING: $g_{p\,max}$*(DC+DW)+$g_{p\,max}$*(EH)+$g_{p\,max}$*(EV)+1.0*(WA)+1.4*(WS)+0.50*(TU)

LOAD	Load Factor	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	1.25	49.30		485.06	0.00		0.00
DW	1.5	1.30		12.77	0.00		0.00
EH	1.425	1.24		24.31	0.00		0.00
EV	1.35	48.97		583.56	0.00		0.00
WA (static)	1.00	-26.84		-363.65	0.00		0.00
WS	1.40	0.00		0.00	0.00		0.00
TU	0.50	0.00		0.00	0.0000		0.0000
SUM		**73.96**		**742.06**	**0.00**		**0.00**

Load Factors Based on this particular LRFD Combination

↓ N/A, Valid for Pile Design Only ↓

10.03 ft	10.03 ft	742.1 k.ft	**-742.1 k.ft**	**74.0 kip**	**0.0 kip**

PIER DESIGN - LOAD COMBINATIONS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	ALH
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Pier	**Date:**	June 25, 2014

LRFD Load Combinations Cont.

Load Combinations : 3.3

LC6 - STRENGTH III SLIDING: $g_{p\,min}$*(DC+DW)+$g_{p\,max}$*(EH)+$g_{p\,min}$*(EV)+1.0*(WA)+1.4*(WS)+0.50*(TU)

LOAD	Load Factor	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	0.90	35.49		349.24	0.00		0.00
DW	0.65	0.56		5.53	0.00		0.00
EH	1.43	1.24		24.31	0.00		0.00
EV	1.00	36.27		432.27	0.00		0.00
WA	1.00	-26.84		-363.65	0.00		0.00
WS	1.40	0.00		0.00	0.00		0.00
TU	0.50	0.00		0.00	0.0000		0.0000
SUM		46.72		447.71	0.00		0.00

Load Factors Based on this particular LRFD Combination

↓ N/A, Valid for Pile Design Only ↓

9.58 ft	9.58 ft	447.7 k.ft	-447.7 k.ft	46.7 kip	0.0 kip

LC7 - STRENGTH V BEARING: $g_{p\,max}$*(DC+DW)+$g_{p\,max}$*(EH)+$g_{p\,max}$*(EV)+1.35*(LL+IM+PL+BR+LS)+1.0*(WA)+0.4*(WS)+1.0*(WL)+0.50*(TU)

LOAD	Load Factor	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	1.25	49.30		485.06	0.00		0.00
DW	1.5	1.30		12.77	0.00		0.00
EH	1.43	1.24		24.31	0.00		0.00
EV	1.35	48.97		583.56	0.00		0.00
LL+IM+PL+BR+LS	1.35	13.24		130.27	1.29		33.55
WA	1.00	-26.84		-363.65	0.00		0.00
WS	0.40	0.00		0.00	0.00		0.00
WL	1.00	0.00		0.00	0.00		0.00
TU	0.50	0.00		0.00	0.0000		0.0000
SUM		87.20		872.33	1.29		33.55

Load Factors Based on this particular LRFD Combination

↓ N/A, Valid for Pile Design Only ↓

10.00 ft	10.00 ft	872.3 k.ft	-838.8 k.ft	87.2 kip	1.3 kip

LC8 - STRENGTH V SLIDING: $g_{p\,min}$*(DC+DW)+$g_{p\,max}$*(EH)+$g_{p\,min}$*(EV)+1.35*(LL+IM+PL+BR+LS)+1.0*(WA)+0.4*(WS)+1.0*(WL)+0.50*(TU)

LOAD	Load Factor	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	0.9	35.49		349.24	0.00		0.00
DW	0.65	0.56		5.53	0.00		0.00
EH	1.425	1.24		24.31	0.00		0.00
EV	1	36.27		432.27	0.00		0.00
LL+IM+PL+BR+LS	1.35	13.24		130.27	1.29		33.55
WA	1.00	-26.84		-363.65	0.00		0.00
WS	0.40	0.00		0.00	0.00		0.00
WL	1.00	0.00		0.00	0.00		0.00
TU	0.50	0.00		0.00	0.0000		0.0000
SUM		59.96		577.98	1.29		33.55

Load Factors Based on this particular LRFD Combination

↓ N/A, Valid for Pile Design Only ↓

9.64 ft	9.64 ft	578.0 k.ft	-544.4 k.ft	60.0 kip	1.3 kip

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	ALH
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Pier	**Date:**	June 25, 2014

LRFD Load Combinations Cont.

Load Combinations : 3.4

LC9 - EXTREME EVENT I BEARING: $g_{p\ max}$*(DC+DW)+$g_{p\ max}$*(EH)+$g_{p\ max}$*(EV)+$g_{EQ\ MAX}$*(LL+IM+PL+BR+LS)+1.0*(EQ)

LOAD	Load Factor	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	1.25	49.30		485.06	0.00		0.00
DW	1.5	1.30		12.77	0.00		0.00
EH	0.00	0.00		0.00	0.00		0.00
EV	1.35	48.97		583.56	0.00		0.00
LL+IM+PL+BR+LS	0.50	4.90		48.25	0.48		12.43
WA	0.00	0.00		0.00	0.00		0.00
EQ	1.00	14.05		276.41	40.30		846.75
SUM		**118.51**		**1406.05**	**40.78**		**859.17**

Load Factors Based on this particular LRFD Combination

↓ N/A, Valid for Pile Design Only ↓

11.86 ft	11.86 ft	1406.1 k.ft	**-546.9 k.ft**	**118.5 kip**	**40.8 kip**

LC10 - EXTREME EVENT I SLIDING: $g_{p\ min}$*(DC+DW)+$g_{p\ max}$*(EH)+$g_{p\ min}$*(EV)+$g_{EQ\ MIN}$*(LL+IM+PL+BR+LS)+1.0*(WA)+1.0*(EQ)

LOAD	Load Factor	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	0.9	35.49		349.24	0.00		0.00
DW	0.65	0.56		5.53	0.00		0.00
EH	0.00	0.00		0.00	0.00		0.00
EV	1.00	36.27		432.27	0.00		0.00
LL+IM+PL+BR+LS	0.00	0.00		0.00	0.00		0.00
WA (seismic)	1.00	-26.84		-363.65	0.00		0.00
EQ	1.00	14.05		276.41	40.30		846.75
SUM		**59.53**		**699.80**	**40.30**		**846.75**

Load Factors Based on this particular LRFD Combination

↓ N/A, Valid for Pile Design Only ↓

11.76 ft	11.76 ft	699.8 k.ft	**146.9 k.ft**	**59.5 kip**	**40.3 kip**

PIER DESIGN - LOAD COMBINATIONS

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	ALH
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Pier	**Date:**	June 25, 2014

LRFD Load Combinations Cont.

Load Combinations : 3.4

LC11 - EXTREME EVENT II BEARING: $g_{p\ max}$*(DC+DW)+$g_{p\ max}$*(EH)+$g_{p\ max}$*(EV)+$g_{EQ\ MAX}$*(LL+IM+PL+BR+LS)+1.0*(EQ)

LOAD	Load Factor	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	1.25	49.30		485.06	0.00		0.00
DW	1.5	1.30		12.77	0.00		0.00
EH	1.43	1.24		24.31	0.00		0.00
EV	1.35	48.97		583.56	0.00		0.00
LL+IM+PL+BR+LS	0.50	4.90		0.00	0.48		12.43
WA	0.00	0.00		0.00	0.00		0.00
CT	1.00	9.81		0.00	0.00		0.00
SUM		**115.51**		**1105.71**	**0.48**		**12.43**

Load Factors Based on this particular LRFD Combination

↓ N/A, Valid for Pile Design Only ↓

9.57 ft	9.57 ft	1105.7 k.ft	**-1093.3 k.ft**	**115.5 kip**	**0.5 kip**

LC12 - EXTREME EVENT II SLIDING: $g_{p\ min}$*(DC+DW)+$g_{p\ max}$*(EH)+$g_{p\ min}$*(EV)+$g_{EQ\ MIN}$*(LL+IM+PL+BR+LS)+1.0*(WA)+1.0*(EQ)

LOAD	Load Factor	Vertical Force (Kips)	Arm (Feet)	Resisting Moment (Ft x K)	Horiz Force (Kips)	Arm (Feet)	Overturn Moment (Ft x K)
DC	0.9	35.49		349.24	0.00		0.00
DW	0.65	0.56		5.53	0.00		0.00
EH	1.43	1.24		24.31	0.00		0.00
EV	1.00	36.27		432.27	0.00		0.00
LL+IM+PL+BR+LS	0.50	4.90		0.00	0.00		0.00
WA (seismic)	1.00	-26.84		-363.65	0.00		0.00
CT	1.00	9.81		0.00	0.00		0.00
SUM		**61.43**		**447.71**	**0.00**		**0.00**

Load Factors Based on this particular LRFD Combination

↓ N/A, Valid for Pile Design Only ↓

7.29 ft	7.29 ft	447.7 k.ft	**-447.7 k.ft**	**61.4 kip**	**0.0 kip**

PIER DESIGN

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	ALH
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Pier	**Date:**	June 25, 2014

References:

- AASHTO LRFD Bridge Design Specifications, 6th Edition, 2012
- ACI 318-08 Building Code Requirements for Structural Concrete, 2005
- 2009 MassDOT LRFD Bridge Manual, including draft Novemeber 2012 provisions
- AASHTO Guide Specifications for LRFD Seismic Bridge Design 2011

Notes: **This template assumes that the soils strata behind the abutment is uniform (only 1 strata is considered).**

Check Bearing Resistance (per AASHTO 11.6.3.2) -- ON SOIL

Stability : 1.0

If supported on soil, the vertical stress (σ_v) shall be calculated assuming a uniformly distributed pressure (V) over an effective base area (B-2e). AASHTO Fig 11.6.3.2-1 ----> qr / Φβ = q_n =

If supported on rock, the vertical stress (σ_v) shall be calculated assuming a linearly distributed pressure over an effective base area. AASHTO Fig 11.6.3.2-2 ----> qr / Φβ = q_n =

Nominal Bearing Resistance, qn:	q_n =	26.00	ksf		
Strength Bearing Resistance Factor, Φβ (AASHTO Table 10.5.5.2.2):		0.45		qr = Φβ * q_n = 11.70	ksf
Extreme Event Bearing Resistance Factor, Φβ (AASHTO 10.5.5.3.3):		1.00		qr = Φβ * q_n = 26.00	ksf

	LOAD COMBINATION	Vertical Force (Kips)	Resisting Moment (Ft x K)	Overturn Moment (Ft x K)	Mnet (Ft x K)	Eccentricty from Toe, et=Mnet/V (Ft)	Eccentricty from CL, e=B/2-et (Ft)	σ_v on soil (ksf)	$\sigma_{v\,max}$ on rock (ksf)	$\sigma_{v\,min}$ on rock (ksf)	$\sigma_v < \Phi\beta * q_n$	
Strength	LC1	84.35	961.54	47.34	914.20	10.84	1.00	4.77	5.59	2.98	OK	
Strength	LC2	103.88	1153.62	47.34	1106.27	10.65	0.81	5.75	6.58	3.97	OK	
Bearing	LC3	91.12	910.93	43.49	867.44	9.52	0.32	4.79	5.08	4.18	OK	
Sliding	LC4	63.88	616.58	43.49	573.09	8.97	0.87	3.56	4.11	2.39	N/A	<--*N/A Sliding Combination
Bearing	LC5	73.96	742.06	0.00	742.06	10.03	0.19	3.83	3.98	3.54	OK	
Sliding	LC6	46.72	447.71	0.00	447.71	9.58	0.26	2.44	2.56	2.19	N/A	<--*N/A Sliding Combination
Bearing	LC7	87.20	872.33	33.55	838.78	9.62	0.22	4.53	4.73	4.13	OK	
Sliding	LC8	59.96	577.98	33.55	544.43	9.08	0.76	3.30	3.75	2.34	N/A	<--*N/A Sliding Combination
Ex. Bearing	LC9	118.51	1406.05	859.17	546.88	4.61	5.23	12.84	17.12	0.00	OK	
Ex. Sliding	LC10	59.53	699.80	846.75	-146.94	-2.47	12.31	**	**	**	N/A	<--*N/A Ex. Sliding Combination
Ex. Bearing	LC11	115.51	1105.71	12.43	1093.28	9.46	0.38	6.10	6.54	5.20	OK	
Ex. Sliding	LC12	61.43	447.71	0.00	447.71	7.29	2.55	4.21	5.55	0.69	N/A	<--*N/A Ex. Sliding Combination

*** Sliding Load Combinations are Not Applicable for checking the Bearing**

**** Eccentricity is such that the resultant vertical force falls outside the footing, hence bearing pressure cannot be calculated.**

PIER DESIGN

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	ALH
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Pier	**Date:**	June 25, 2014

Check Overturning (per AASHTO 11.6.3.3) -- ON SOIL

Stability : 2.0

e allowable (ftgs on soil): 6.56 ft

e allowable (ftgs on rock): 8.86 ft

If e < e allowable, Overturning is OK:

	LOAD COMBINATION	Eccentricty from CL, e=B/2-et (Ft)	Check Overturning	
Strength	**LC1**	1.00	OK	
Strength	**LC2**	0.81	OK	
Bearing	**LC3**	0.32	OK	
Sliding	**LC4**	0.87	N/A	**<--*N/A Sliding Combination**
Bearing	**LC5**	0.19	OK	
Sliding	**LC6**	0.26	N/A	**<--*N/A Sliding Combination**
Bearing	**LC7**	0.22	OK	
Sliding	**LC8**	0.76	N/A	**<--*N/A Sliding Combination**
Ex. Bearing	**LC9**	5.23	OK	
Ex. Sliding	**LC10**	12.31	N/A	**<--*N/A Ex. Sliding Combination**
Ex. Bearing	**LC11**	0.38	OK	
Ex. Sliding	**LC12**	2.55	N/A	**<--*N/A Ex. Sliding Combination**

*** Sliding Load Combinations are Not Applicable for checking Overturning**

PIER DESIGN

General Information

Project Number:	1298\127-1298-12001-LT0077	**Designed By:**	ALH
Description:	Khost Bridge No. 10	**Checked By:**	SAM
Structure:	Pier	**Date:**	June 25, 2014

Check Sliding (per AASHTO 10.6.3.4)

Stability : 3.0

Ignore Passive Resistance of Soil per MassHighway

Strength Sliding Resistance Factor, Φ_τ (AASHTO Table 11.5.7-1):	1.00	
Extreme Event Sliding Resistance Factor, Φ_ϖ (AASHTO 10.5.5.3.3):	1.00	
Internal Friction Angle of Drained Soil, Φ_f:	33.00	degrees
tan δ: = tan Φ_f (per AASHTO 10.6.3.4-2):	0.65	for concrete against soil. Multiply by 0.8 for precast concrete footing

	LOAD COMBINATION	Vertical Force (Kips)	Rt = V * tan δ: (Kips)	Φ_τ (Strength) Φ_ω (Extreme) (Kips)	Nom. Sliding Resistance Φ_τ*Rt (Kips)	Horiz Force (Kips)	Check Sliding	
Strength	**LC1**	84.35	54.78	1.00	54.78	3.37	N/A	**<--*N/A Strength Combination**
Strength	**LC2**	103.88	67.46	1.00	67.46	3.37	N/A	**<--*N/A Strength Combination**
Bearing	**LC3**	91.12	59.17	1.00	59.17	1.68	N/A	**<--*N/A Bearing Combination**
Sliding	**LC4**	63.88	41.49	1.00	41.49	1.68	OK	
Bearing	**LC5**	73.96	48.03	1.00	48.03	0.00	N/A	**<--*N/A Bearing Combination**
Sliding	**LC6**	46.72	30.34	1.00	30.34	0.00	OK	
Bearing	**LC7**	87.20	56.63	1.00	56.63	1.29	N/A	**<--*N/A Bearing Combination**
Sliding	**LC8**	59.96	38.94	1.00	38.94	1.29	OK	
Ex. Bearing	**LC9**	118.51	76.96	1.00	76.96	40.78	N/A	**<--*N/A Ex. Bearing Combination**
Ex. Sliding	**LC10**	59.53	38.66	1.00	38.66	40.30	NO GOOD	
Ex. Bearing	**LC11**	115.51	75.01	0.65	48.71	0.00	N/A	**<--*N/A Ex. Bearing Combination**
Ex. Sliding	**LC12**	61.43	39.89	0.65	25.91	0.00	OK	

Results Summary:

Stability : 4.0

STABILITY RESULTS:

LOAD COMBINATION:	BEARING RESISTANCE	OVERTURNING	SLIDING	
LC1	**OK**	**OK**	N/A	<== Construction
LC2	**OK**	**OK**	N/A	<== Construction
LC3	**OK**	**OK**	N/A	
LC4	N/A	N/A	**OK**	
LC5	**OK**	**OK**	N/A	
LC6	N/A	N/A	**OK**	
LC7	**OK**	**OK**	N/A	
LC8	N/A	N/A	**OK**	
LC9	**OK**	**OK**	N/A	
LC10	N/A	N/A	**NO GOOD**	
LC11	**OK**	**OK**	N/A	
LC12	N/A	N/A	**OK**	

Appendix C

Cost Calculations

MODIFIED STRUCTURE ALTERNATE – COMPARISON

SUPERSTRUCTURE

SUPERSTRUCTURE (LBG)

– 2 SPAN SLAB BRIDGE (SPAN = 12.17 M)

CONCRETE	L M	W M	T M	V M3
DECK/SLAB	24.34	10.95	0.65	173.24

Σ 173.24
SAY 175 M^3

REBAR

RATIO = 200 #/CY

REBAR → 175 M^3 ≈ 229 CY

229 CY * 200 #/CY = 45,800 lbs
≈ 20,800 Kg ≈ 20.8 M.TON

SUPERSTRUCTURE SUMMARY (LBG)

CONCRETE = 175 M^3
REBAR = 20.8 METRIC TON

TETRA TECH
One Grant Street
Framingham, MA 01701-9005
(508) 903-2000

JOB Khost 10
SHEET NO. 1 OF
CALCULATED BY alh DATE 7/1/14
CHECKED BY SAM DATE 7/7/14
SCALE

MODIFIED STRUCTURE ALTERNATE – COMPARISON

SUPERSTRUCTURE (ALTERNATE)
- 2 SPAN R.C BEAMS (SPAN = 16.8 M)

CONCRETE	L M	W M	T M	V M^3	QTY	V M^3
DECK	33.6	10.95	0.23	84.6	1	84.6 SAY 85.0
BEAM	16.8	0.6	1.5	15.12	12	181.44 SAY 182

Σ 267 M^3

REBAR (ALT)

	DECK	BEAM	UNIT	TOTAL
CONCRETE	85	182	M3	
	112	239	CY	
RATIO	200	150	#/CY	
REBAR	22,400	35,850	lbs	
	≈ 10,200	16,300	Kgs	26,500
	10.2	16.3	M.TON	26.5

SUPERSTRUCTURE SUMMARY (ALTERNATE)

CONCRETE = 267 M^3
REBAR = 26.5 METRIC TONS

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SHEET NO. 2 OF
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MODIFIED STRUCTURE ALTERNATE – COMPARISON

ELASTOMERIC BEARINGS

BEARINGS (LBG)

Ø

BEARINGS (ALTERNATE)

OF SPANS = 2
BEAMS / SPAN = 6
BEARINGS / BEAM = 2
TOTAL BEARINGS = 24

ELASTOMERIC BEARINGS SUMMARY

	QTY
LBG	0
ALTERNATE	24

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JOB KHOST 10
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MODIFIED STRUCTURE ALTERNATE - COMPARISON

SUBSTRUCTURE

SUBSTRUCTURE - LBG (2010)

CONCRETE (LBG)

	L M	W M	D M	V M^3	QTY	V M^3
ABUTMENT						
FTG	10.95	6.0	1.2	78.8	2	157.6
STEM	10.95	1.2	8.35	109.7	2	219.4
						Σ 377
PIER						
FTG	12.95	6.0	1.5	116.6	1	116.6
STEM	10.95	1.0	6.4	70.1	1	70.1
						Σ 186.7
TOTAL						563.7 M^3
					SAY	575 M^3

REBAR (LBG)

CONCRETE	FTG	STEM	UNIT	TOTAL
ABUT.	158	220	M^3	
PIER	117	71	M^3	
Σ VOL	275	291	M^3	
	≈ 360	≈ 381	CY	
RATIO	120	100	#/CY	
REBAR	43,200	38,100	lbs	81,300
	≈ 19,600	17,300	kg	36,900
	≈ 19.6	17.3	METRIC TON	36.9

SUBSTRUCTURE SUMMARY (LBG)

CONCRETE = 575 M^3
REBAR = 36.9 METRIC TONS

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JOB KHOST 10. MODIFIED ALT (SUBSTRUCTURE)
SHEET NO. 4 OF
CALCULATED BY aln DATE 7/1/14
CHECKED BY SAM DATE 7/7/14
SCALE

MODIFIED STRUCTURE ALTERNATE - COMPARISON

SUBSTRUCTURE (CONT.)

SUBSTRUCTURE (ALTERNATE)

CONCRETE (ALT)

	L M	W M	D M	V M^3	QTY	V M^3
ABUT						
FTG	13.45	7.0	1.5	141.2	2	282.4
STEM	10.95	2.2	6.01	144.8	2	289.6
Σ						Σ 572.0
PIER						
FTG	13.45	5.6	1.5	113	1	113
STEM	11.4	1.5	7.7	131.7	1	131.7
Σ						Σ 244.7
						816.7
					SAY	825.0

REBAR (ALTERNATE)

CONCRETE	FTG	STEM	UNIT	TOTAL
ABUT	283	290	M^3	
PIER	113	132	M^3	
Σ VOL	396	422	M^3	
	518	552	CY	
RATIO	120	100	#/CY	
REBAR	62,160	55,200	lbs	117,360
≃	28,200	25,100	kgs	53,300 kg
	28.2	25.1	M.TON	53.3 M.TON

SUBSTRUCTURE SUMMARY (ALTERNATE)

CONCRETE = 825. M^3

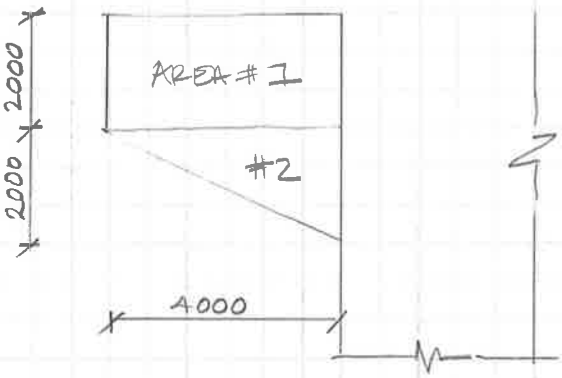
REBAR = 53.3 METRIC TON

TETRA TECH One Grant Street Framingham, MA 01701-9005 (508) 903-2000	JOB KHOST 10 SHEET NO. 5 OF CALCULATED BY aeh DATE 7/1/14 CHECKED BY SAM DATE 7/7/14 SCALE

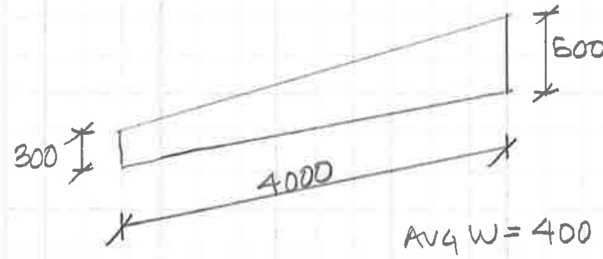
MODIFIED STRUCTURE ALTERNATE- COMPARISON

SUBSTRUCTURE (WINGWALLS)

SUBSTRUCTURE (LBG)



ELEVATION



AVG W = 400 mm

PLAN VIEW

CONCRETE

	L m	W m	D m	V m^3	QTY	V M^3
WINGWALL						
AREA 1	4.0	0.4	2.0	3.2	4	12.8
AREA 2	4.0	0.4	2.0	1.6	4	6.4
						Σ 19.2
						SAY 20.0 M^3

REBAR

RATIO = 100 #/CY

REBAR → 20 M^3 ≈ 27 CY = 2,700 lbs

≈ 1300 kg ≈ 1.3 METRIC TON

SUPERSTRUCTURE WALL SUMMARY (LBG)

CONCRETE = 20.0 M^3

REBAR = 1.3 METRIC TONS

TETRA TECH

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Framingham, MA 01701-9005
(508) 903-2000

JOB KH05T 10: MSA- SUBSTRUCTURE -WALLS

SHEET NO. LBG OF

CALCULATED BY aln DATE 7/1/14

CHECKED BY SAM DATE 7/7/14

SCALE

MODIFIED STRUCTURE ALTERNATE - COMPARISON

SUBSTRUCTURE (WINGWALLS)

SUBSTRUCTURE (ALTERNATE)

CONCRETE (ALTERNATE - REF KHDST No. 9)

WALL	STEM L m	STEM W m	STEM H m	STEM V m³	FTG L m	FTG W m	FTG H m	FTG V m³	TOTAL V m³
SOUTHWEST	7.74	0.85	7.01	46.12	7.74	4.5	1.0	17.42	63.54
NORTHWEST	7.0	0.85	6.26	37.25	7.0	4.5	1.0	15.57	52.82
NORTHEAST	7.0	0.85	6.37	37.90	7.0	4.5	1.0	31.50	69.4
SOUTHEAST	3.5	0.85	7.79	23.18	3.5	4.5	1.0	15.75	38.93
TOTAL				Σ 144.5				Σ 80.2	224.17
			SAY	150			SAY	85	235.0

REBAR (ALTERNATE - REF KHDST NO.9)

	STEM	FTG	UNIT	
WALL VOL.	150	85	M³	
	200	115	CY	
RATIO	100	120	#/CY	
REBAR	20,000	13,800	lbs	
	≈ 9,100	6,300	kg	Σ 15,400
	9.1	6.3	M.TON	Σ 15.4

SUBSTRUCTURE WALL SUMMARY (ALTERNATE)

CONCRETE = 235.0 M³
REBAR = 15.4 M.TONS

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JOB
SHEET NO. OF
CALCULATED BY alh DATE 7/1/14
CHECKED BY SAM DATE 7/7/14
SCALE

MODIFIED STRUCTURE ALTERNATE – COMPARISON

QUANTITY & COST SUMMARY

ITEM	UNIT	UNIT COST	LBG QTY	LBG COST	ALTERNATE QTY	ALTERNATE COST	Δ
SUPERSTRUCTURE							
CONCRETE	M^3	170	175	29,750	267	45,390	+ 15,640
REBAR	M. TON	1450	20.8	30,160	26.5	38,425	+ 8,265
BEARINGS	EA	850	0	0	24	20,400	+ 20,400
							Σ 44,305
SUBSTRUCTURE							
CONCRETE	M^3	170	575	97,750	825	140,250	+ 42,500
REBAR	M. TON	1450	36.9	53,505	53.3	77,285	+ 23,780
							Σ 66,280
WALLS							
CONCRETE	M^3	170	20	3400	235	39,950	+ 36,550
REBAR	M. TON	1450	1.3	1,885	15.4	22,330	+ 20,445
							Σ 56,995

SUMMARY	LBG	ALTERNATE	Δ
TOTAL	$216,450	$ 384,030	+ $ 167,580
ADD ≈ 10%			+ $ 16,800
TOTAL			+ $ 184,380
		SAY →	+ $ 190,000

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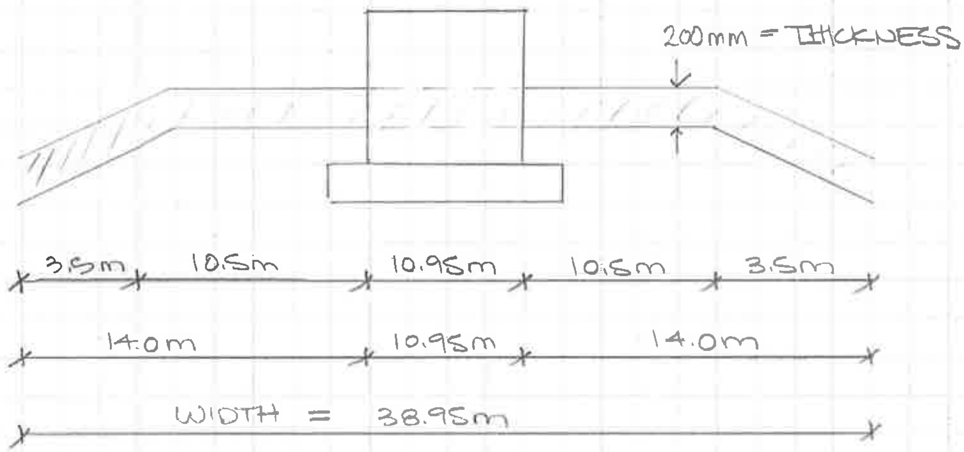
Framingham, MA 01701-9005
(508) 903-2000

JOB KHOST NO. 10 –
SHEET NO. ____ OF ____
CALCULATED BY aeh DATE 7/3/14
CHECKED BY SAM DATE 7/7/14
SCALE ____

CONCRETE SCOUR MATTRESS (FOR LBG DESIGN)



LENGTH = 24.34m + Approx (5m X 2) = 34.34m

APPROACH

CONCRETE

WIDTH	=	38.95m
LENGTH	=	34.34 m
THICKNESS	=	0.20m
VOLUME	=	267.5 m^3
+ 10%	=	26.8 m^3
TOTAL	=	294.3 m^3

SAY → 300 m^3

REBAR

#12 @ 250 (T&B, EACH WAY) ⇒ RATIO 175 #/CY (REF KHOST 9)

300 m^3 = 392.4 CY ⇒ 392.4 CY × 175 #/CY = 68,670 #s

REBAR = 68,670 #'S
+10% = 6,867 #S
TOTAL = 75,537 #'S = 34,263 kg SAY → 35,000 kg (35 METRIC TON)

COST

	UNIT	#	UNIT COST	COST
CONCRETE	M^3	300	\$170/M^3	\$ 51,000
REBAR	TON	35	\$ 1450/TON	\$ 50,750
TOTAL				\$ 101,750
SAY				\$ 105,000

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Framingham, MA 01701-9005
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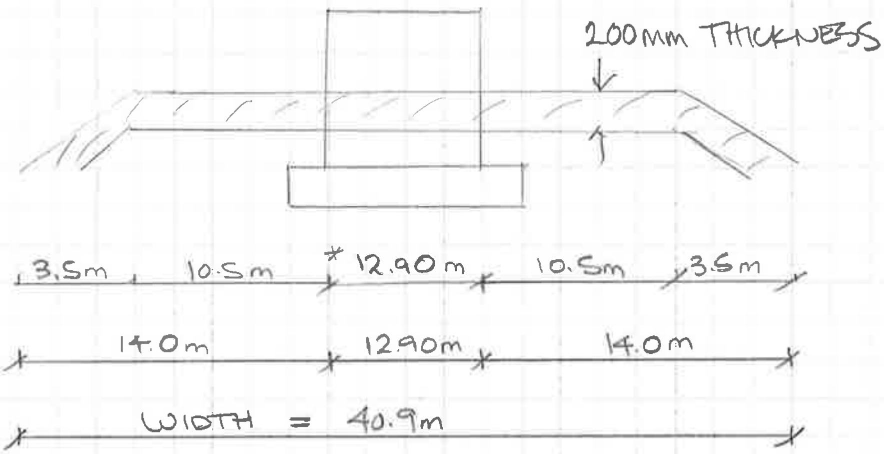
JOB BRIDGE NO. 10 - (LBG)
SHEET NO. MATTRESS OF
CALCULATED BY aeh DATE 7/1/14
CHECKED BY SAM DATE 7/7/14
SCALE

CONCRETE SCOUR MATTRESS (ALTERNATE)



* USED 12.9m = WIDTH OF PIER

LENGTH = 16.8 X 2 SPANS + 7.5m (APPROX WW) X 2 = 48.6M

CONCRETE

WIDTH = 40.9 m
LENGTH = 48.6 m
THICKNESS = 0.20 m
VOLUME = 397.6 m^3
≈ + 10% = 40 m^3
TOTAL = 437.6 m^3 → SAY 450 M^3

REBAR

#12@250 (EACH WAY & T&B) → RATIO 175#/CY (REF KHOST NO 9)

450 m^3 = 588.6 CY → 588.6 CY * 175#/CY = 103,005 lbs

REBAR = 103,005 + 10% (10,300.5) = 113305.5 lbs SAY 51,400 kg
= 51395 kg 51.4 M.TON

COST

	UNIT	#	UNIT COST	COST
CONCRETE	M^3	450	$170	76,500
REBAR	TON	51.4	$1450	74,530
TOTAL				151,030
SAY				$155,000

TETRA TECH

One Grant Street
Framingham, MA 01701-9005
(508) 903-2000

JOB KHOST NO. 10 - Alternate
SHEET NO. Mattress OF
CALCULATED BY aeh DATE
CHECKED BY SAM DATE 7/7/14
SCALE

RAISED ROADWAY ALTERNATE

STATION 51+840 → 52+040

LENGTH = 200m

WIDTH = 10.950m SAY 11m

THICKNESS = 0.75m ← ASSUMED AVG Δ VERTICAL TRANSITION

FILL

LENGTH = 200m
WIDTH = 11m
THICKNESS = 0.75m
VOLUME = 1,650 M^3 → SAY 1700 M^3

EXCAVATION (REMOVE CAUSEWAY)

LENGTH = 30m
WIDTH = 10m
Δ HEIGHT = (TOP) 1820.0 − (CHANNEL MIN BOT) 1815.15 = 4.85m
VOLUME = 1455 M^3 → SAY 1500 M^3

COST

ITEM	UNIT	#	UNIT/$	TOTAL
FILL	M^3	1700	$4.00	6800
EXCAVATION	M^3	1500	$5.00	7500
SUBTOTAL				14,300
SAY				16,000
+ 100% (ALL UNKNOWN EARTHWORK)				16,000
+ 50% (Δ IN ABUT & PIER HEIGHTS)				8,000
				$40,000

* AN ADDITIONAL 50% WAS INCLUDED TO ACCOUNT FOR CHANGE IN ABUTMENT & PIER HEIGHTS DUE TO THE RAISED ROADWAY.

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JOB KHOST NO. 10
SHEET NO. ______ OF ______
CALCULATED BY alh DATE 7/1/14
CHECKED BY SAM DATE 7/7/14
SCALE ______

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